Performance-based Analysis for Implementation of Systematic Rehabilitation of Concrete Hydraulic Structures

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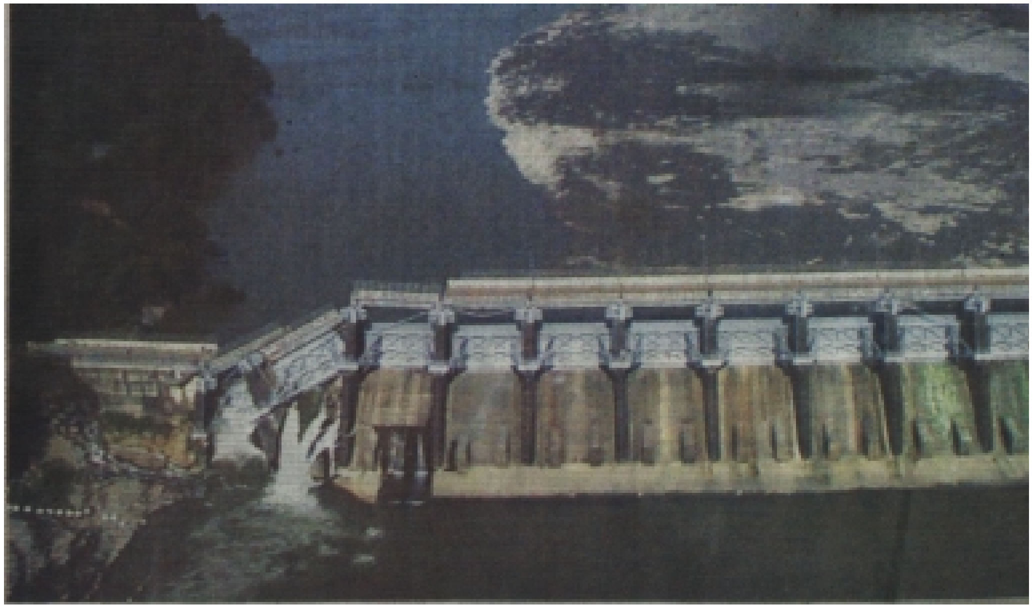
Why Seismic Rehabilitation?

- Earthquake is a real threat
- Save lives
- Reduce the risk of catastrophic failure
- Minimize economic impact
- Minimize the cost of repair
- Minimize the risk of service interruption



Systematic Rehabilitation

Performance-based Analysis

- Reasonable Design/Evaluation Earthquake Motions
- Appropriate Method of Analysis
- Probable Load Combinations
- Material Properties and Damping consistent with existing conditions
- Structural models that account for existing conditions and method of construction
- Performance evaluation in terms of demand-capacity ratios, strength and displacement capacities

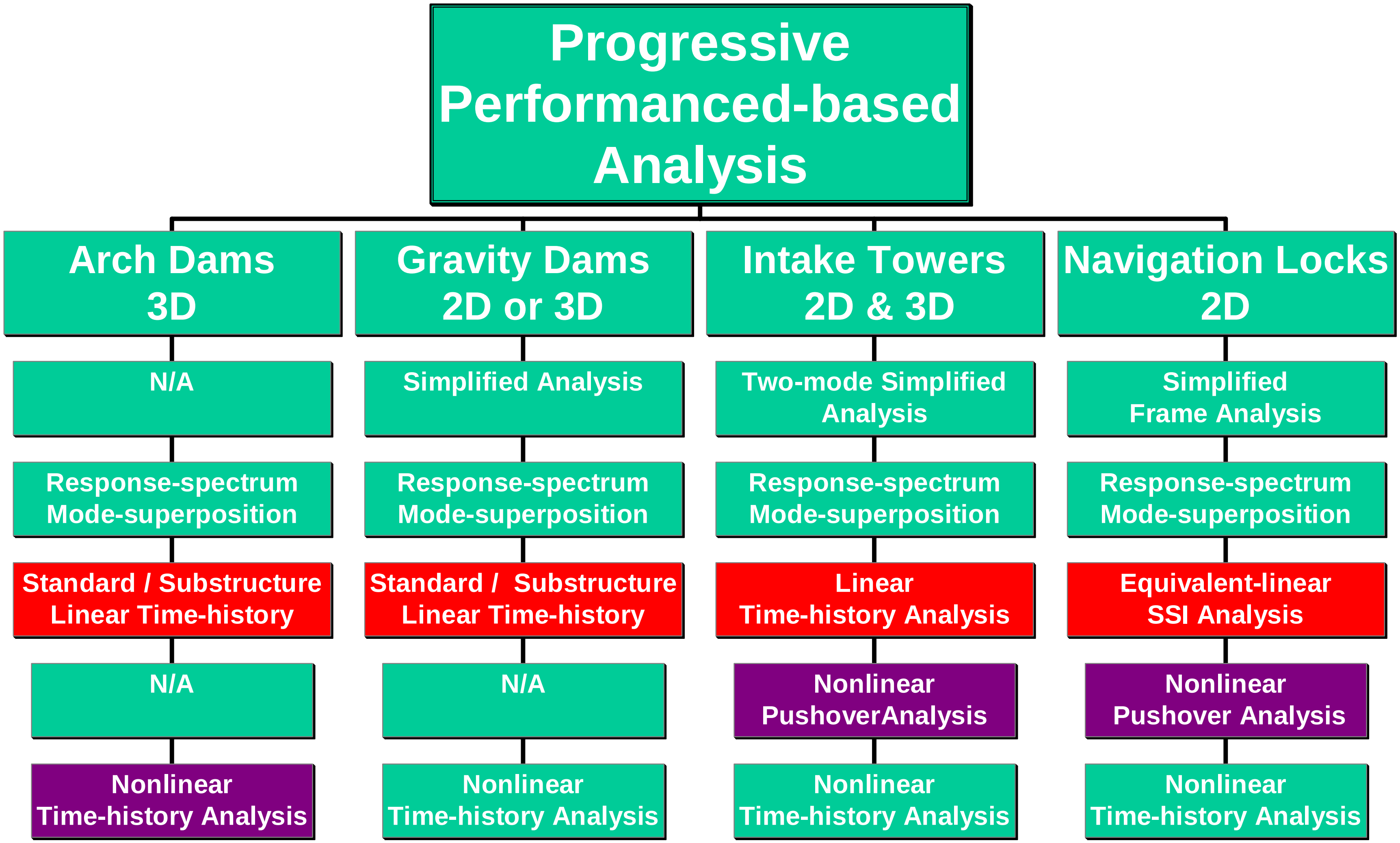
Performance-based Design

- Serviceability Performance

  Serviceable and operable immediately following an OBE event (elastic or/ nearly elastic)
- Damage Control Performance

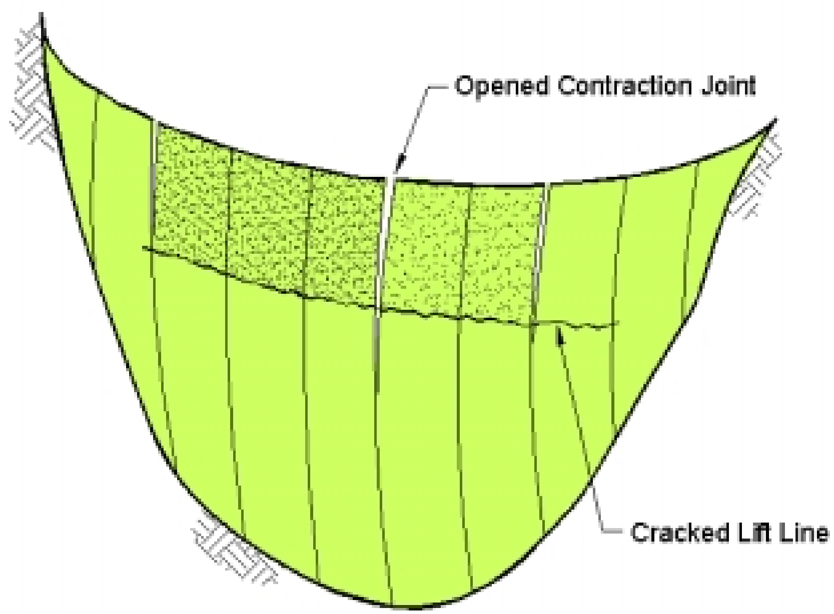
  Limited nonlinear behavior can occur under MDE, if nonlinear displacement demands are low and load resistance is not diminished
- Collapse Prevention Performance

  Collapse of the structure should be prevented regardless of level of damage


Progressive Performanced-based Analysis
Arch Dams
3D
Gravity Dams
2D or 3D
Intake Towers
2D & 3D
Navigation Locks
2D
N/A
Simplified Analysis
Two-mode Simplified Analysis
Simplified Frame Analysis
Response-spectrum Mode-superposition
Response-spectrum Mode-superposition
Response-spectrum Mode-superposition
Response-spectrum Mode-superposition
Standard / Substructure Linear Time-history
Standard / Substructure Linear Time-history
Linear Time-history Analysis
Equivalent-linear SSI Analysis
N/A
N/A
Nonlinear PushoverAnalysis
Nonlinear Pushover Analysis
Nonlinear Time-history Analysis
Nonlinear Time-history Analysis
Nonlinear Time-history Analysis
Nonlinear Time-history Analysis

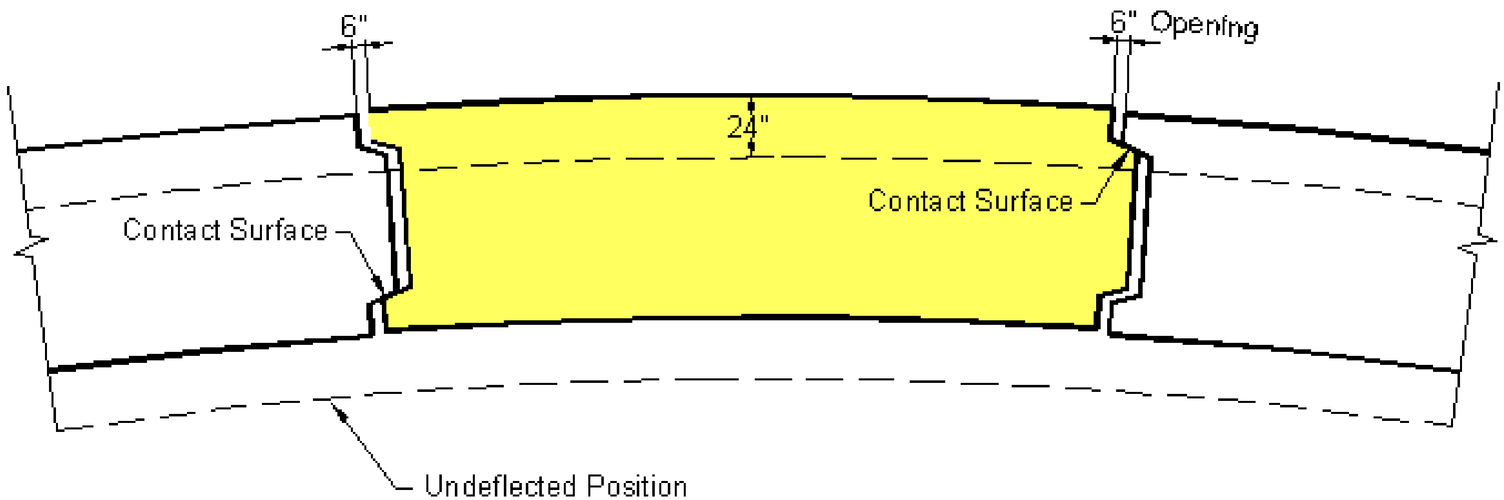
Nonlinear Behavior and Modes of Failure of Arch Dams

- Contraction joints may open and close repeatedly during ground shaking
- Contraction joint opening releases arch tensile stresses and transfers forces to cantilevers
- The increase cantilever stresses may exceed tensile strength of lift lines causing horizontal cracks
- Potentially opened contraction joints and cracked lift lines may subdivide the monolithic arch into one or several partially free cantilever blocks

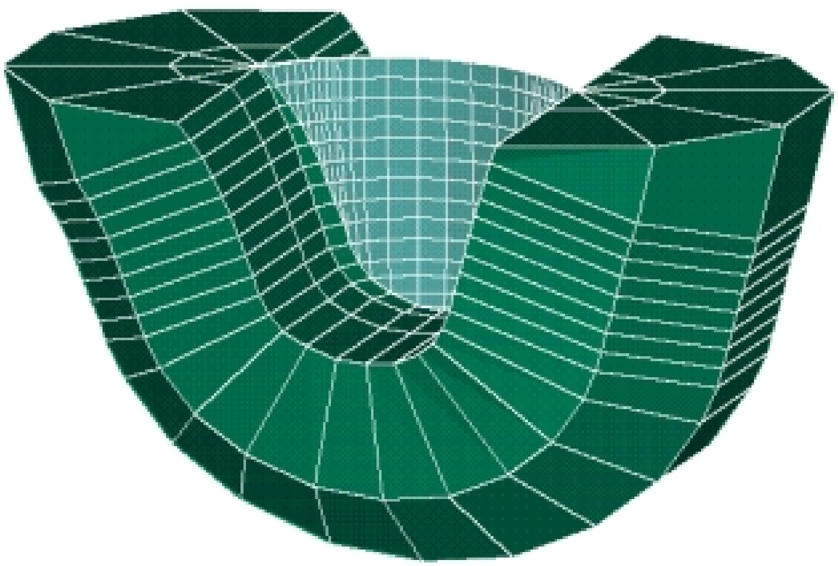


Nonlinear Behavior and Modes of Failure of Arch Dams

- Any failure of the arch dam more likely would involve sliding stability of partially free cantilever blocks
- For small and moderate joint openings, the partially free blocks may remain stable through interlocking (wedging) with adjacent block
- The extent of interlocking depends on the depth and type of shear keys
- The magnitude of compressive stresses, extent of joint opening or cracking, and amplitude of non-recoverable block movements will control the overall stability of the dam, rather than the magnitude of calculated tensile stresses



Dam-Foundation FE Model



Morrow Point Dam-Foundation Model

- **Dam**:

  Shell/Solid Elements

- **Foundation**:

  Massless model:

  – Accounts for flexibility only

  – Fixed boundaries

  Viscoelastic Half-space:

  – Assumes homogenous rock and infinitely long canyon

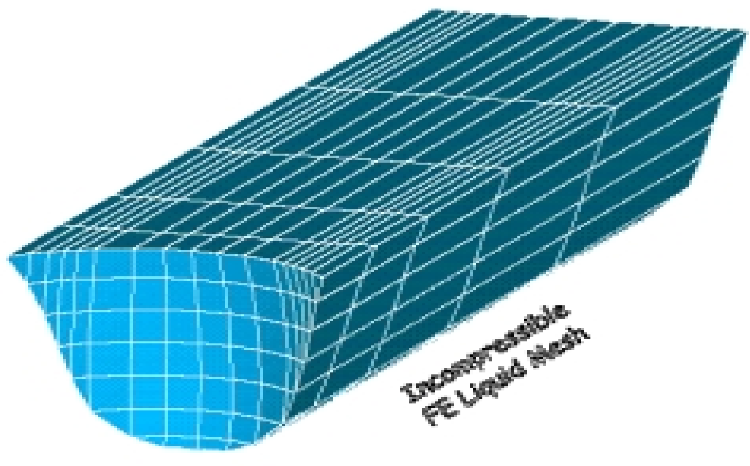
  – Accounts for inertia, damping and flexibility

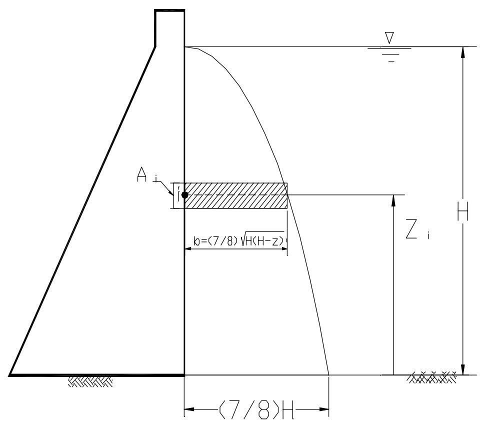
  – Treated as 2D boundary problem

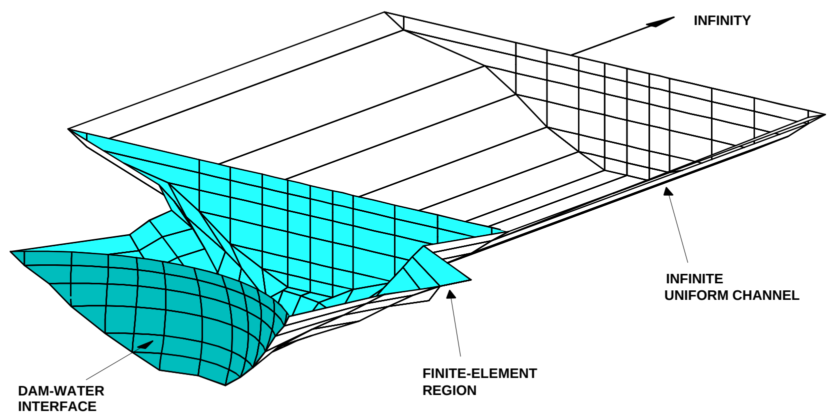
  – Leads to impedance matrix at the interface

Dam-Water Interaction Model

- Generalized Westergaard Added Mass
- FE solution of wave equation for incompressible water
- FE solution of wave equation for compressible water with absorptive boundaries



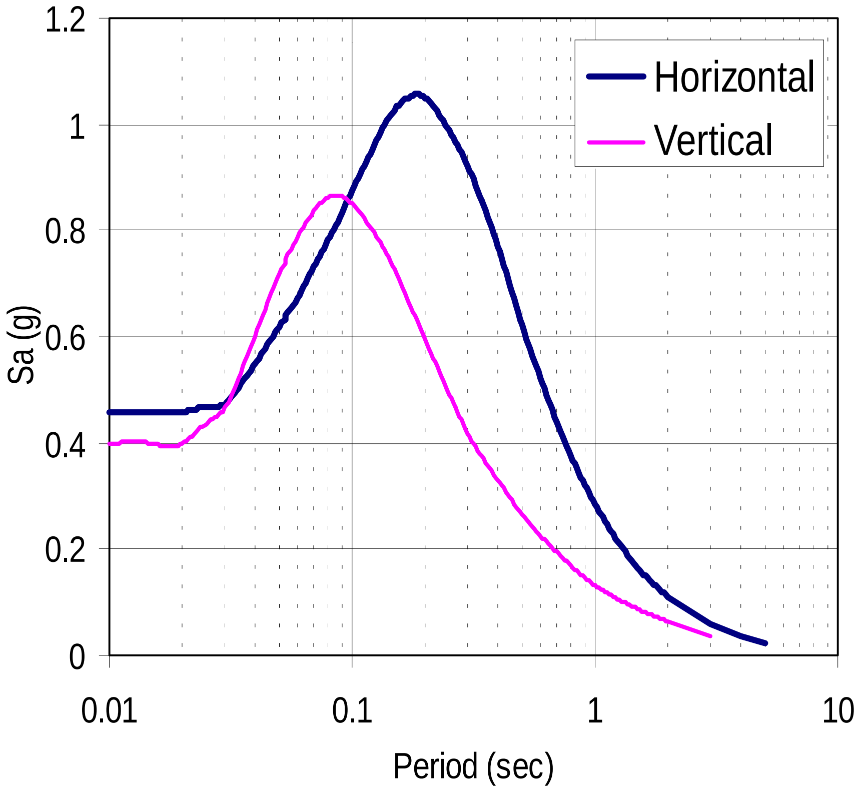




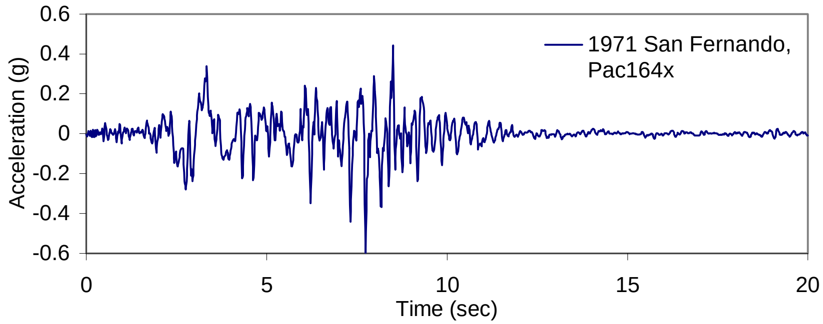
Earthquake ground Motion

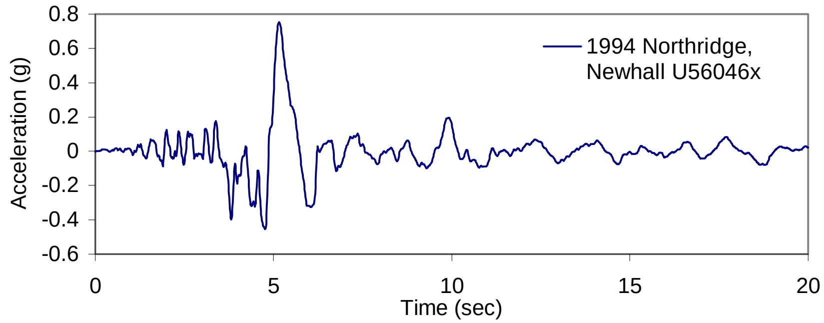
Near-source Earthquake Records

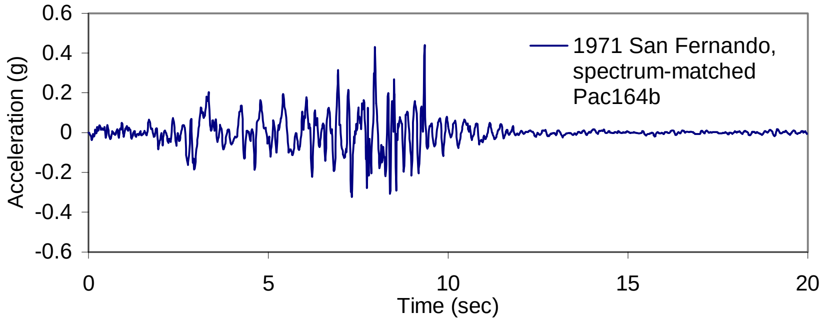
Earthquake Records	Name	Scale
Pacoima Dam, downstream record **1971 San Fernando earthquake** **M_w 6.6, R = 2.8 km**	Pacx	0.52
Spectrum-matched **1971 Pacoima Dam record**	Pacb	1.00
Pacoima Dam, downstream record **1994 Northridge earthquake** **M_w 6.7, R = 8 km**	Pacn	1.13
Newhall, West Pico Canyon Boulevard **1994 Northridge earthquake** **M_w 6.7, R = 7.1 km**	U56	1.80
Coyote Lake Dam **1984 Morgan Hill earthquake** **M_w 6.2, R = 0.1 km**	Cld	0.64
Gilroy Array No. 1 **1989 Loma Prieta earthquake,** **M_w 6.9, R = 11 km**	Gly	0.81

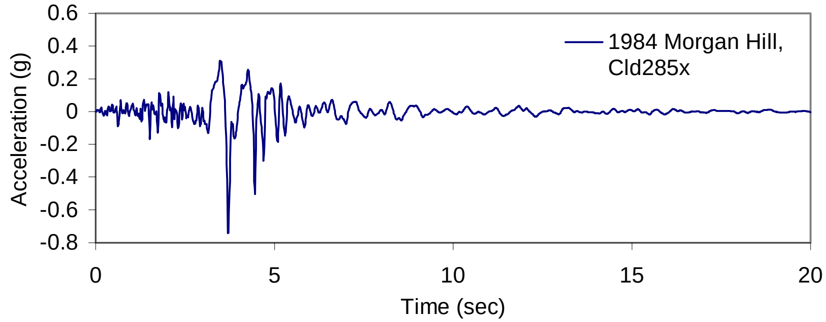


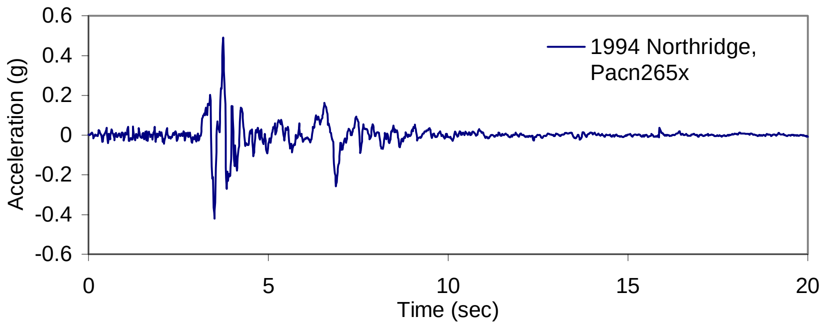
Input Acceleration Time Histories

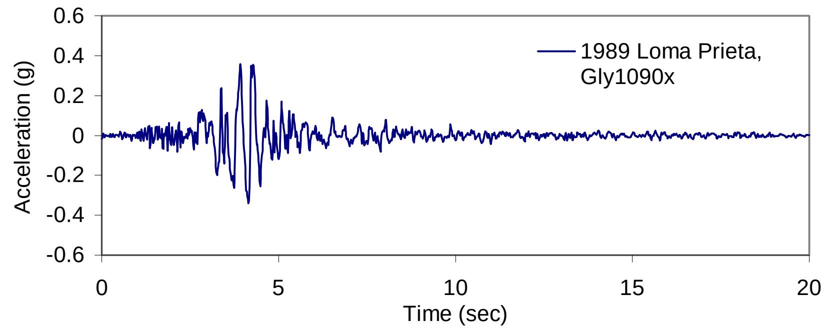




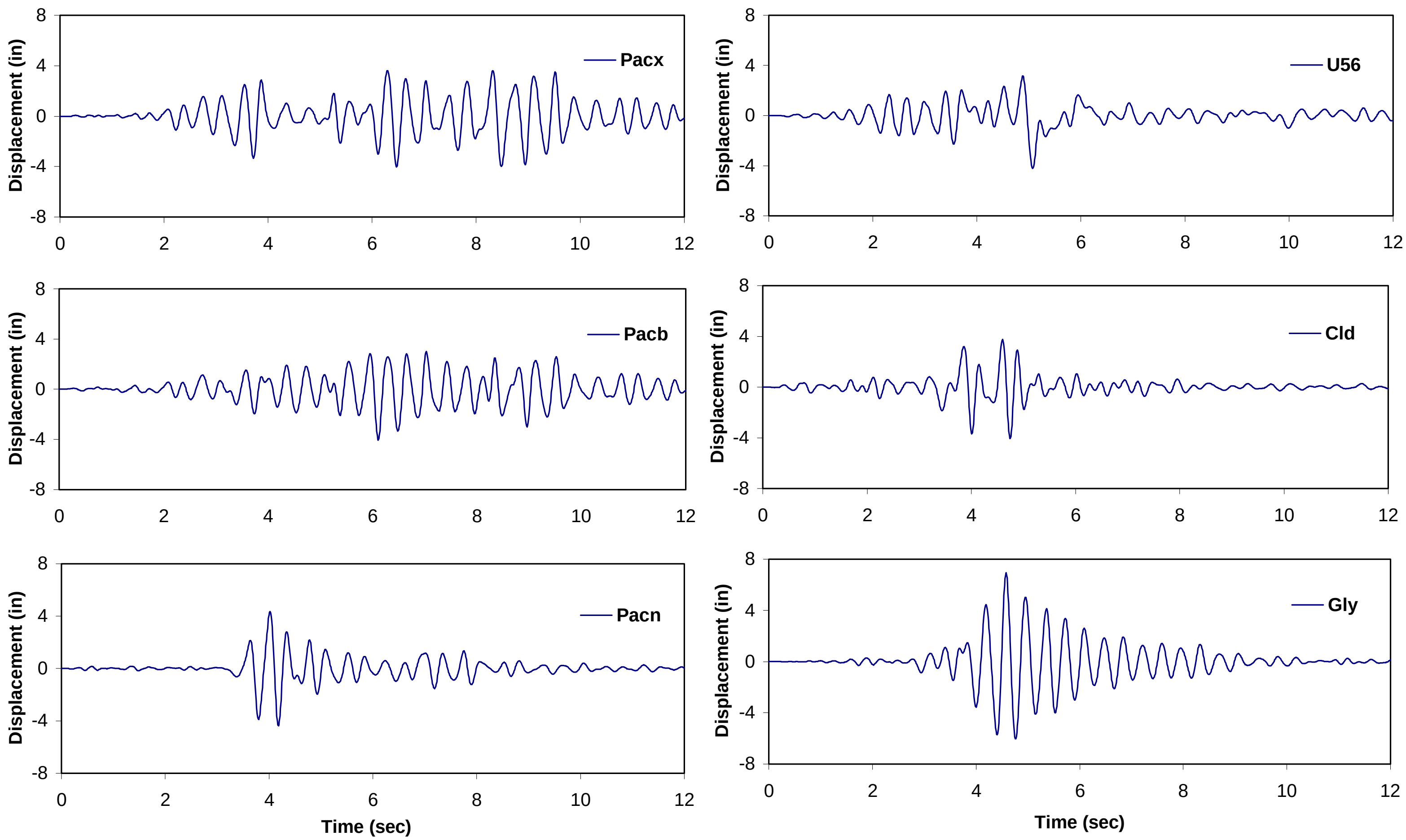




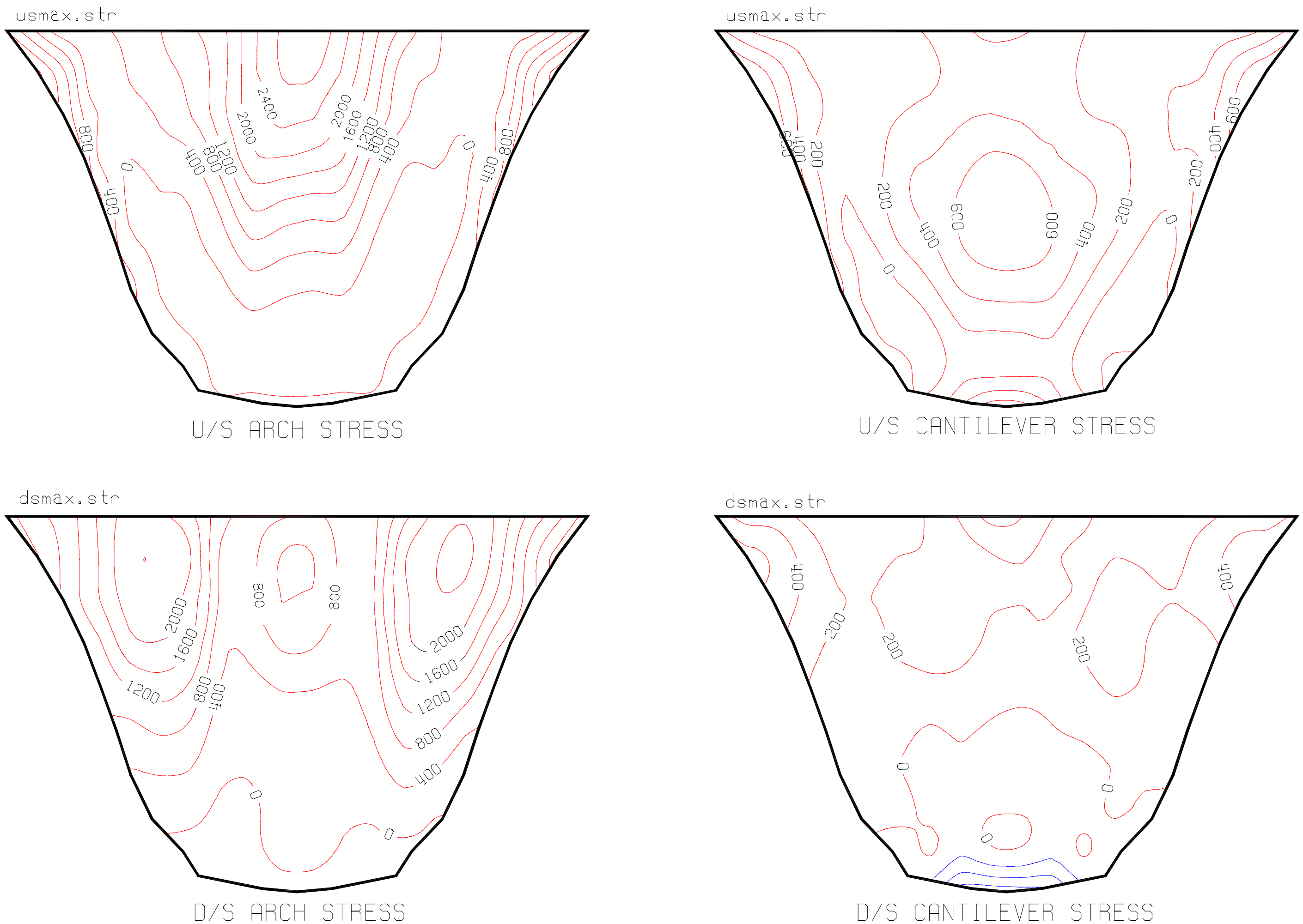




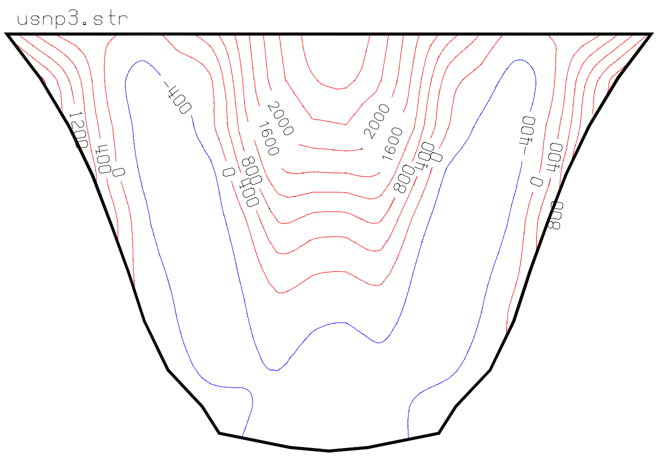
Time Histories of Crest Displacement



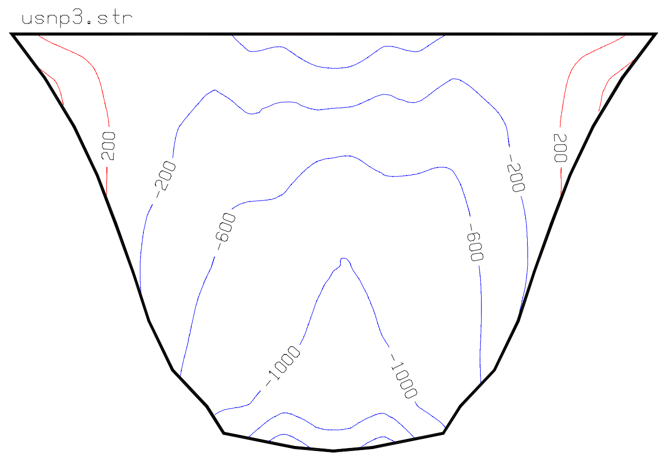
Envelope of Maximum Stresses (Gilroy)



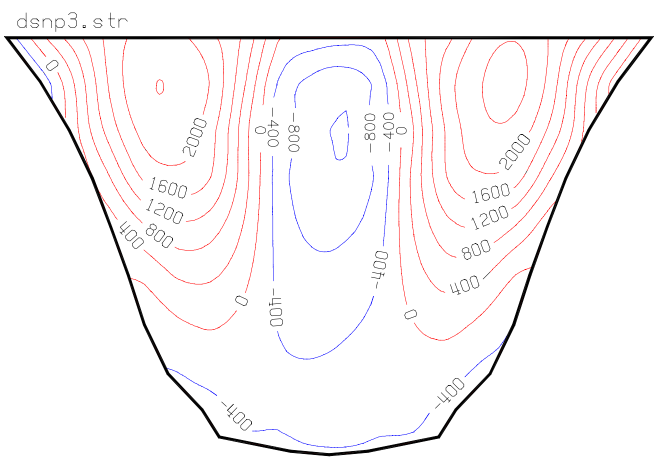
Concurrent Stresses at the Time of Maximum Arch Stress



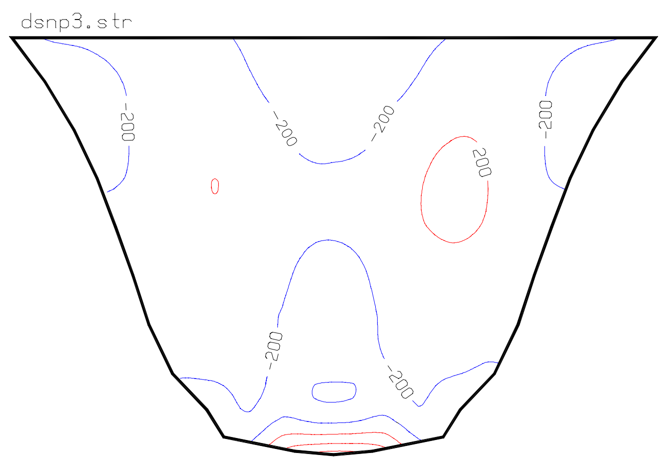
U/S ARCH STRESS



U/S CANTILEVER STRESS

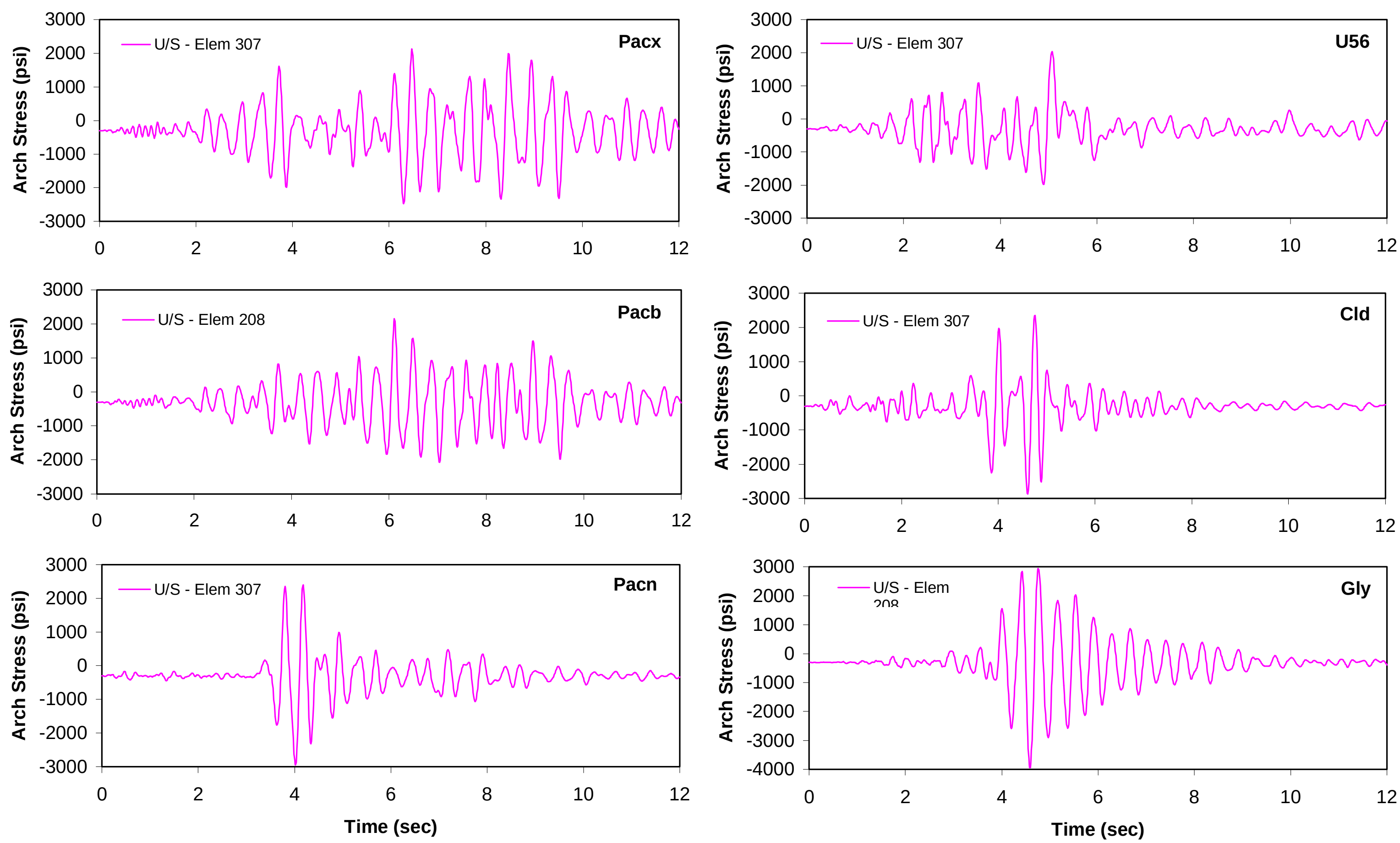


D/S ARCH STRESS

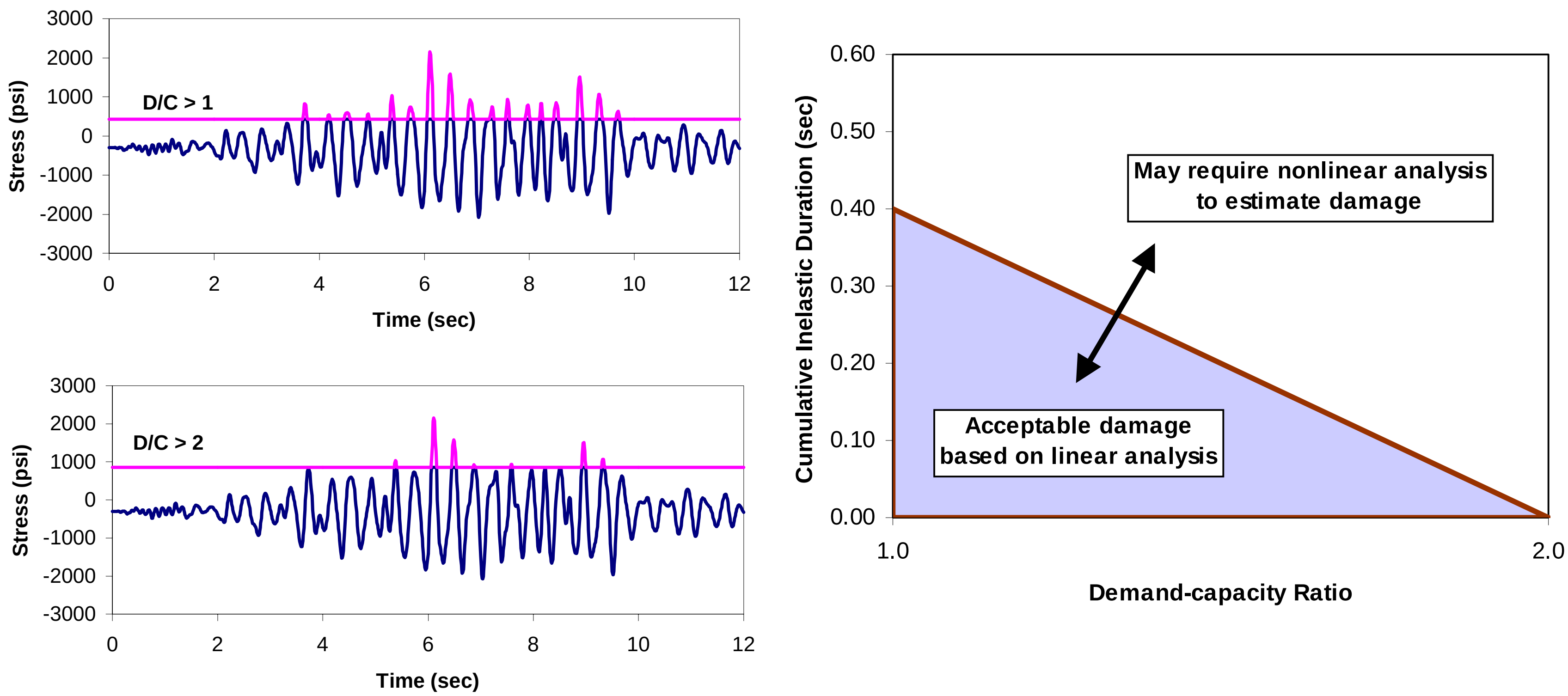


D/S CANTILEVER STRESS

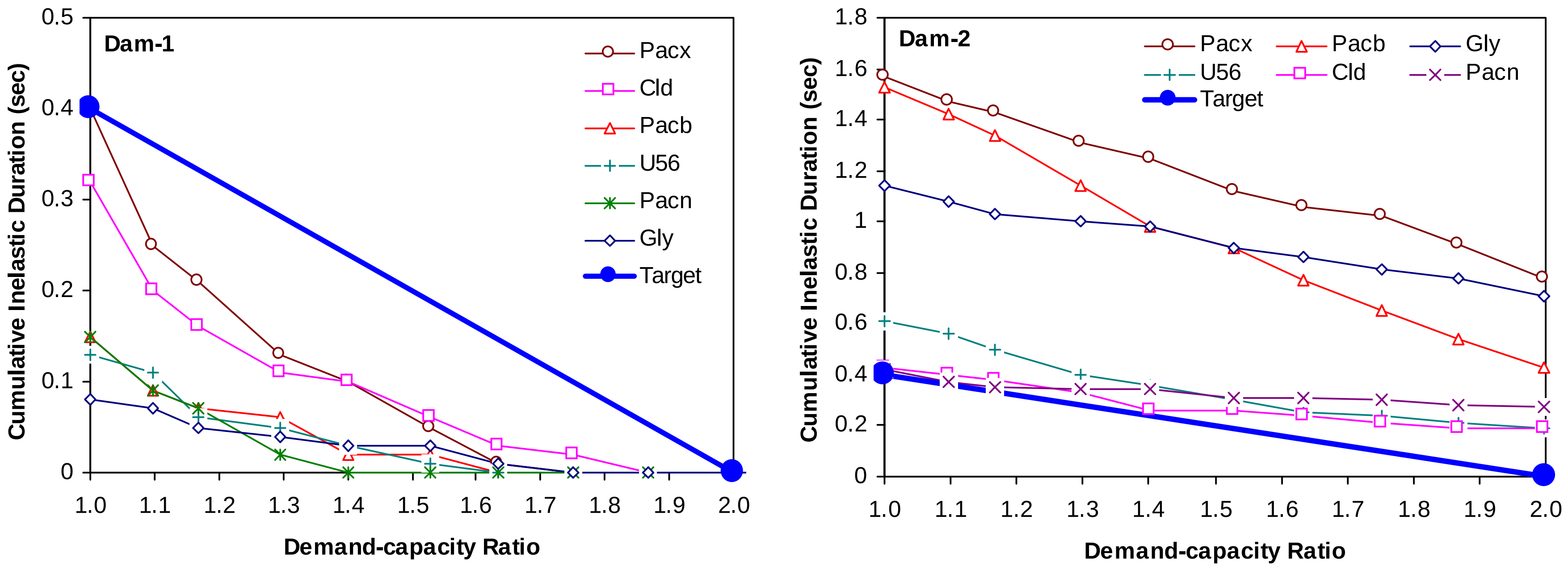
Time History of maximum Arch Stresses



Damage Criteria for Linear Analysis
(Acceptable Performance for Arch Dams)

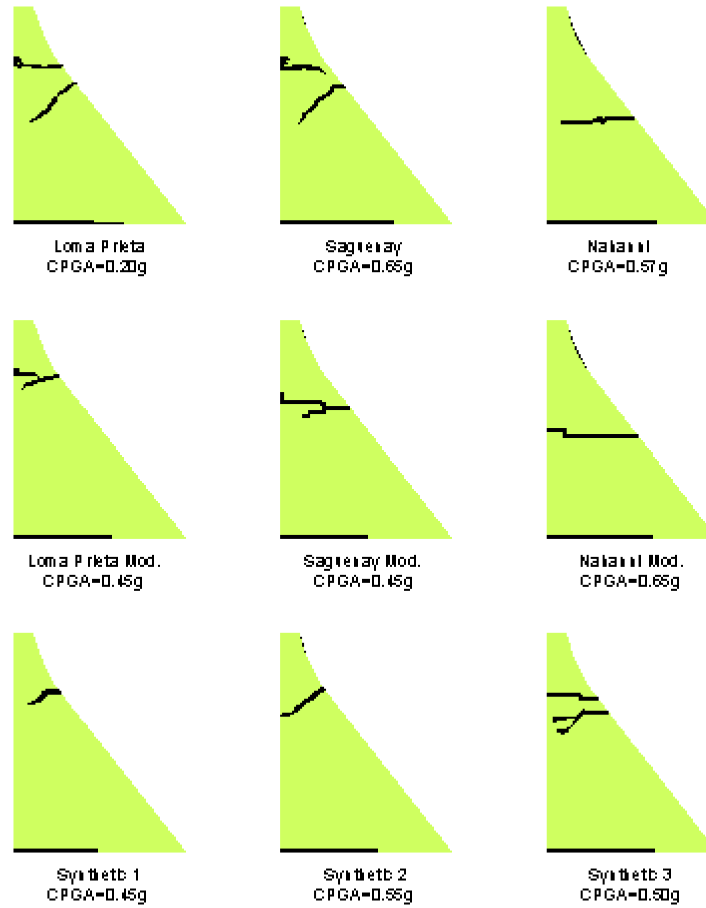


Damage Criteria for Arch Dams (Acceptable Performance)

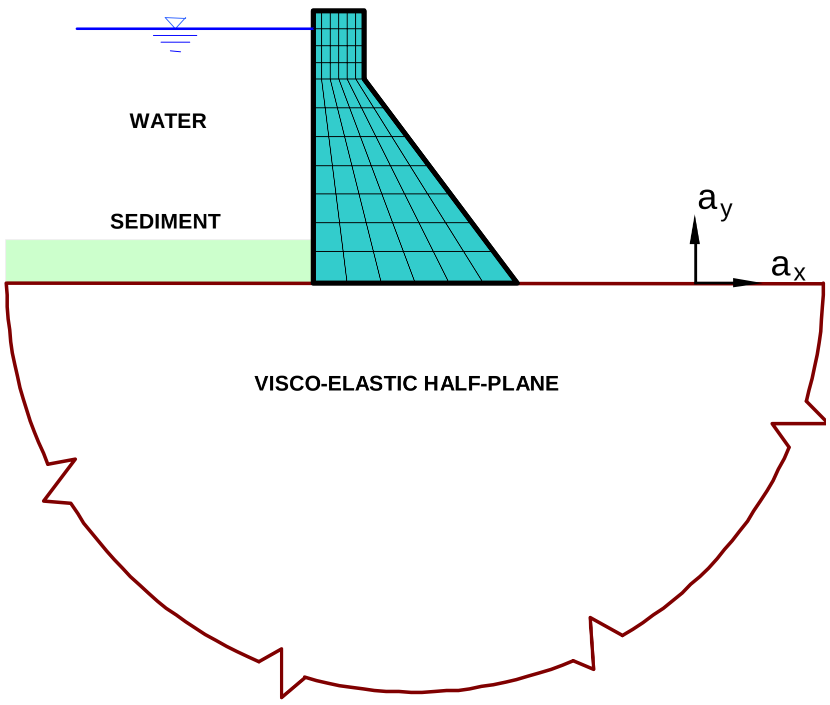


Nonlinear Behavior and Modes of Failure of Gravity Dams

- Formation, location, extent, and orientation of tensile cracking are sensitive to characteristics of the earthquake ground motion
- Cracking always initiates at the base of the dam
- Cracks at the top generally initiate from the D/S face and are horizontal or sloping downward
- A crack sloping down from the D/S is more stable against sliding than a crack with a upward slope
- Any failure would likely involve sliding along the cracked surfaces



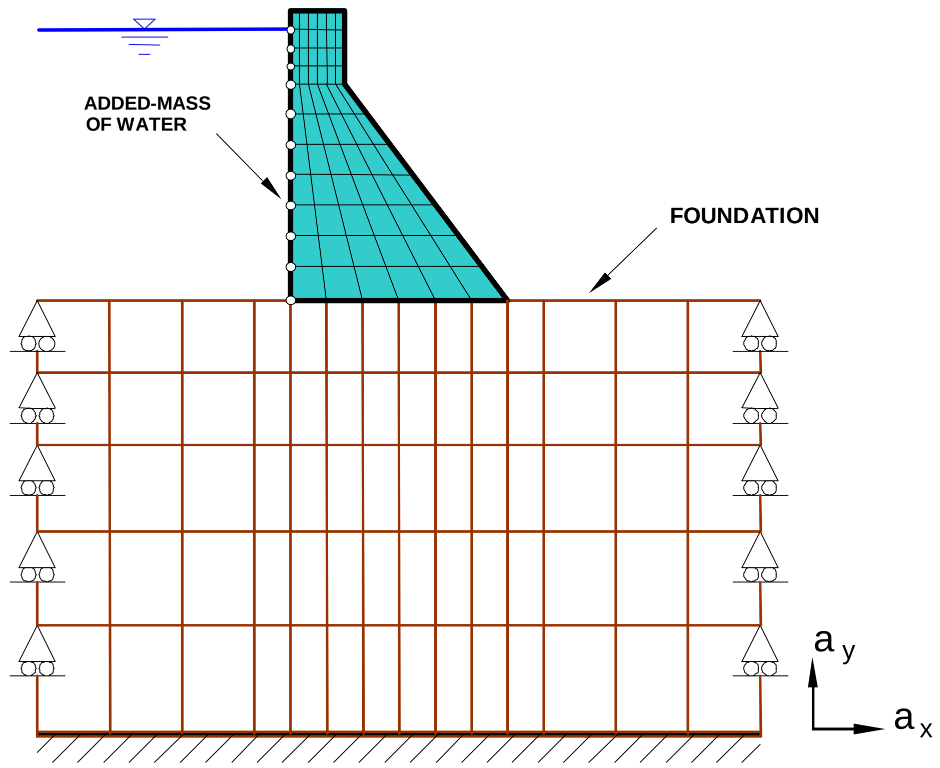
Sub-structure FE Model of Gravity Dam



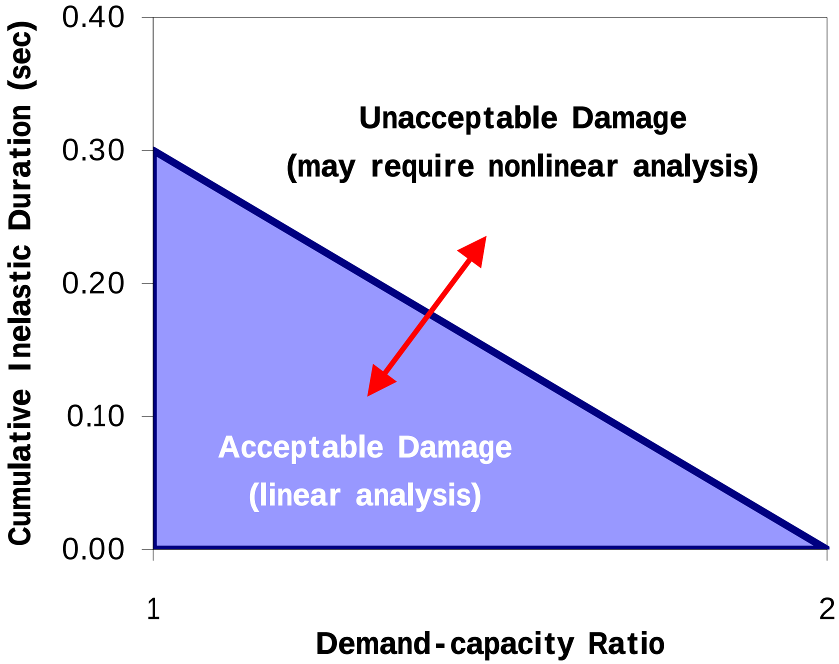
- Complete system is divided into 3 substructures - - dam, water, and foundation rock
- Dam is modeled using standard FE method
- Water is idealized as a continuum leading to frequency-dependent hydrodynamic forces
- Foundation region is idealized as continuum resulting in dynamic stiffness (impedance) matrix

Standard FE Model of Gravity Dam

- <u>Complete system</u> of dam, water, and foundation is idealized and analyzed as a single composite model
- <u>Dam</u> is modeled using standard FE method
- <u>Water</u> is represented by Westergaard added mass
- <u>Foundation region</u> is represented by a FE mesh accounting for flexibility only



Damage Criteria for Linear Analysis
(Acceptable Performance for Gravity Dams)



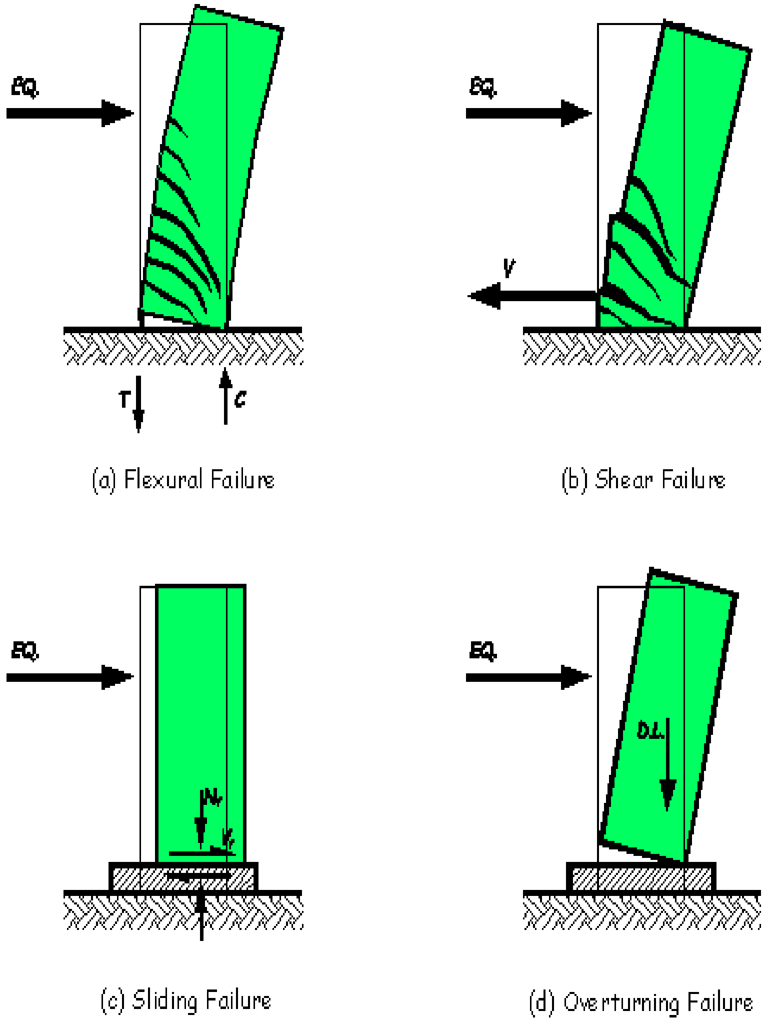
- D/C<1, linear elastic response
- Damage Acceptable if

  - D/C <2
  - Duration below the curve
  - Overstressed region<15% of dam surface area
- Otherwise

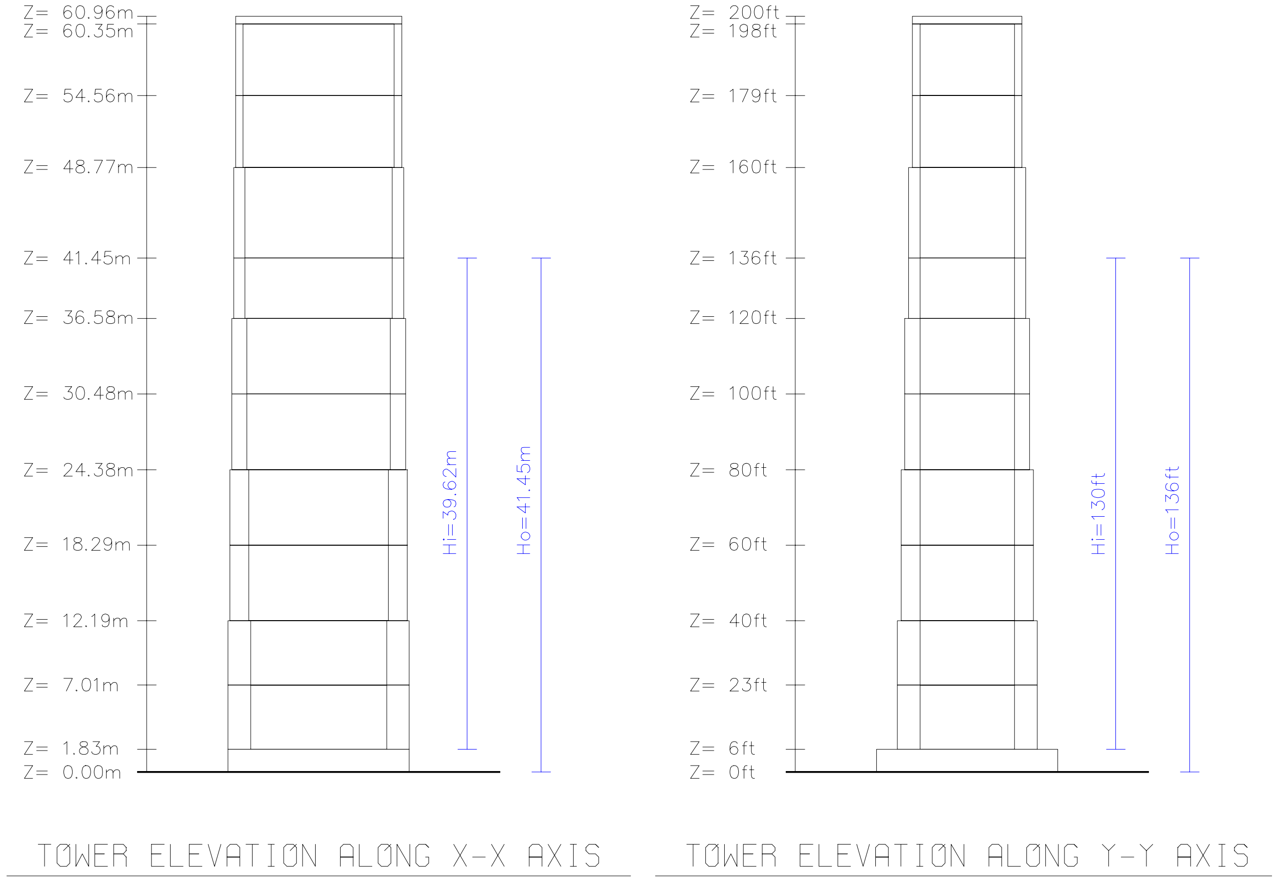
  May require nonlinear analysis or retrofit

Modes of Failure of Freestanding Towers

- Different combinations and sequence of failures (a), (b), (c), and (d) are also possible
- Flexure is desired mode of nonlinear behavior offering energy dissipation through inelastic deformation
- Shear failure should be avoided due to small energy dissipation and rapid strength degradation (non-ductile)

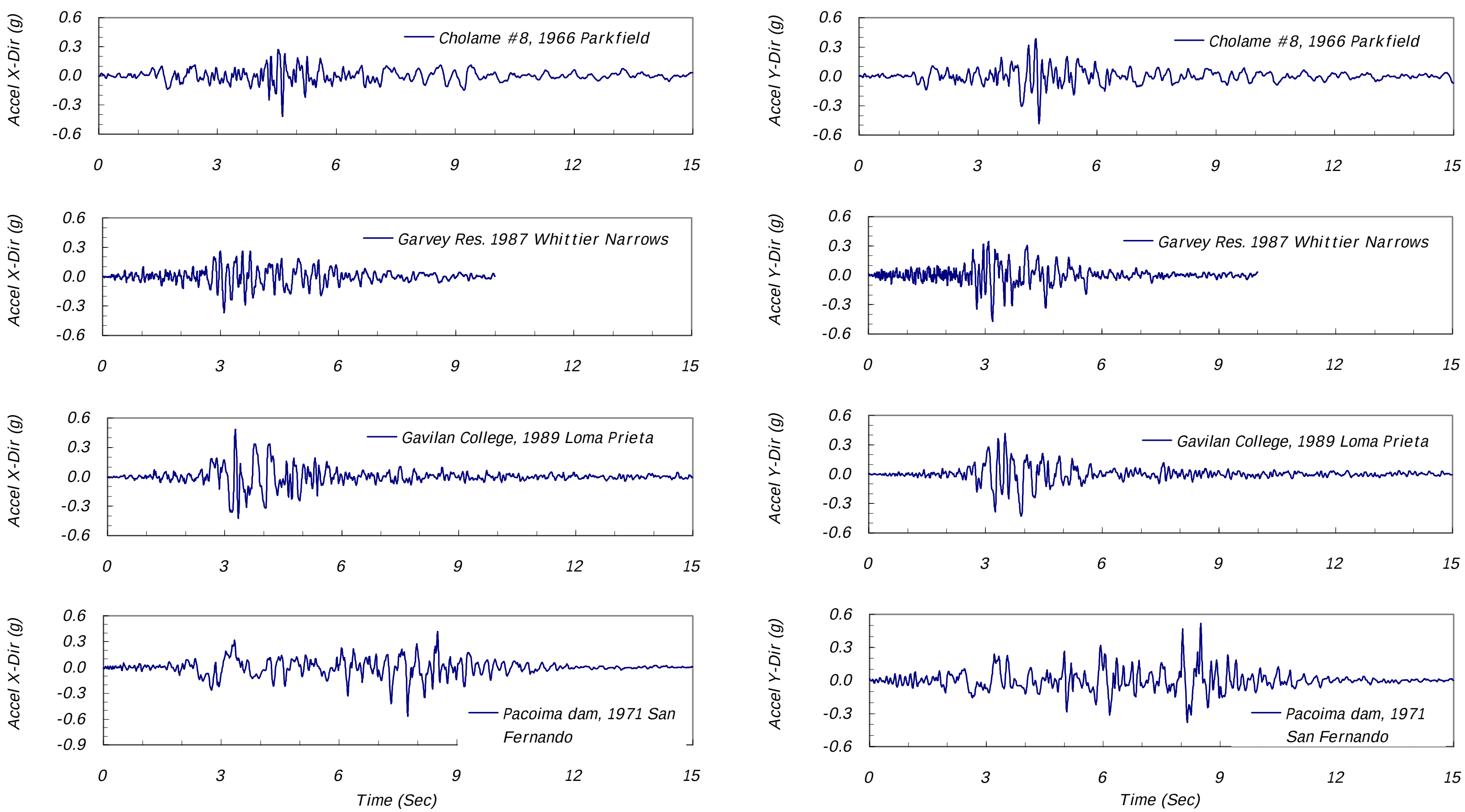


Example Intake Tower



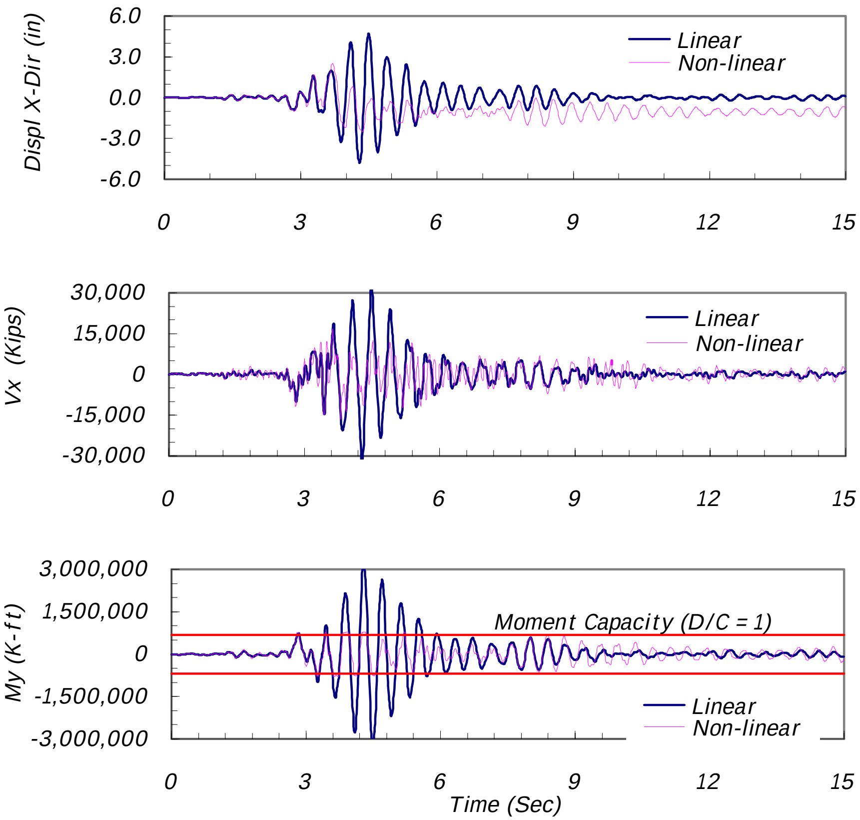
Input Acceleration Time Histories

(M_w 6.5 at 5 km)

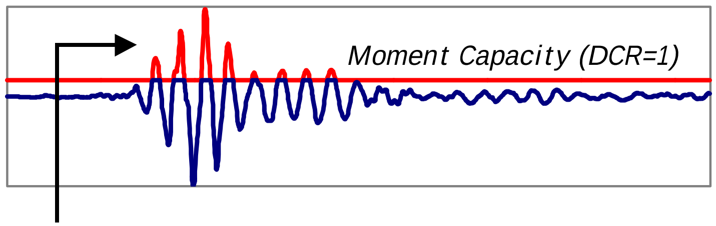


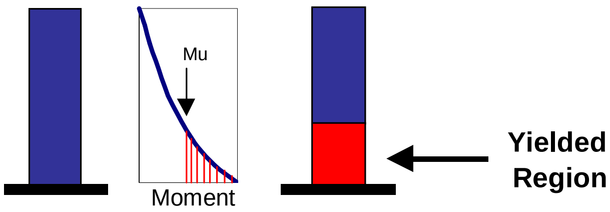
Maximum Response of Example Tower

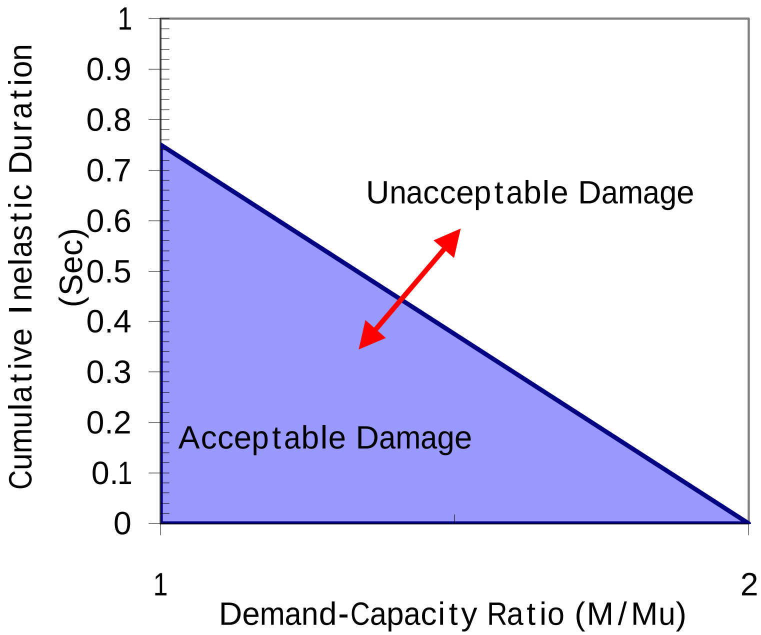
TH#	Earthquake Record	Maximum Displ. (in)	Maximum Moment (k-ft)	Maximum Shear (Kips)
X-Component of Earthqauke Ground Motion (X-Direction)				
1	1966 Parkfield	1.95	1,351,870	13,149
2	1987 Whittier Narrows	2.14	1,453,510	15,296
3	1989 Loma Prieta	4.80	3,376,720	33,308
4	1971 San Fernando	4.28	2,779,910	28,977
Y-Component of Earthqauke Ground Motion (Y-Direction)				
1	1966 Parkfield	3.05	1,518,030	24,293
2	1987 Whittier Narrows	2.49	1,014,640	14,410
3	1989 Loma Prieta	4.56	2,224,440	25,426
4	1971 San Fernando	5.94	2,394,360	21,734

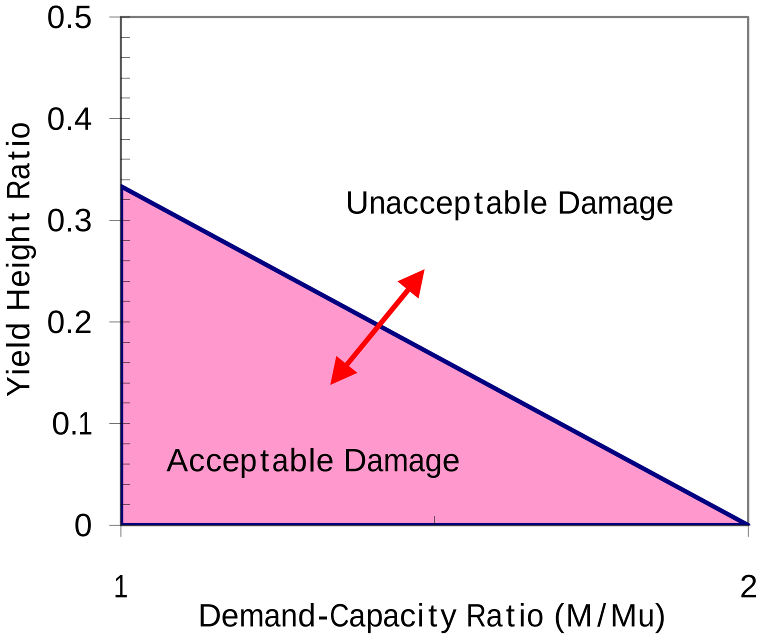


Damage Criteria for Linear Analysis of Intake Towers

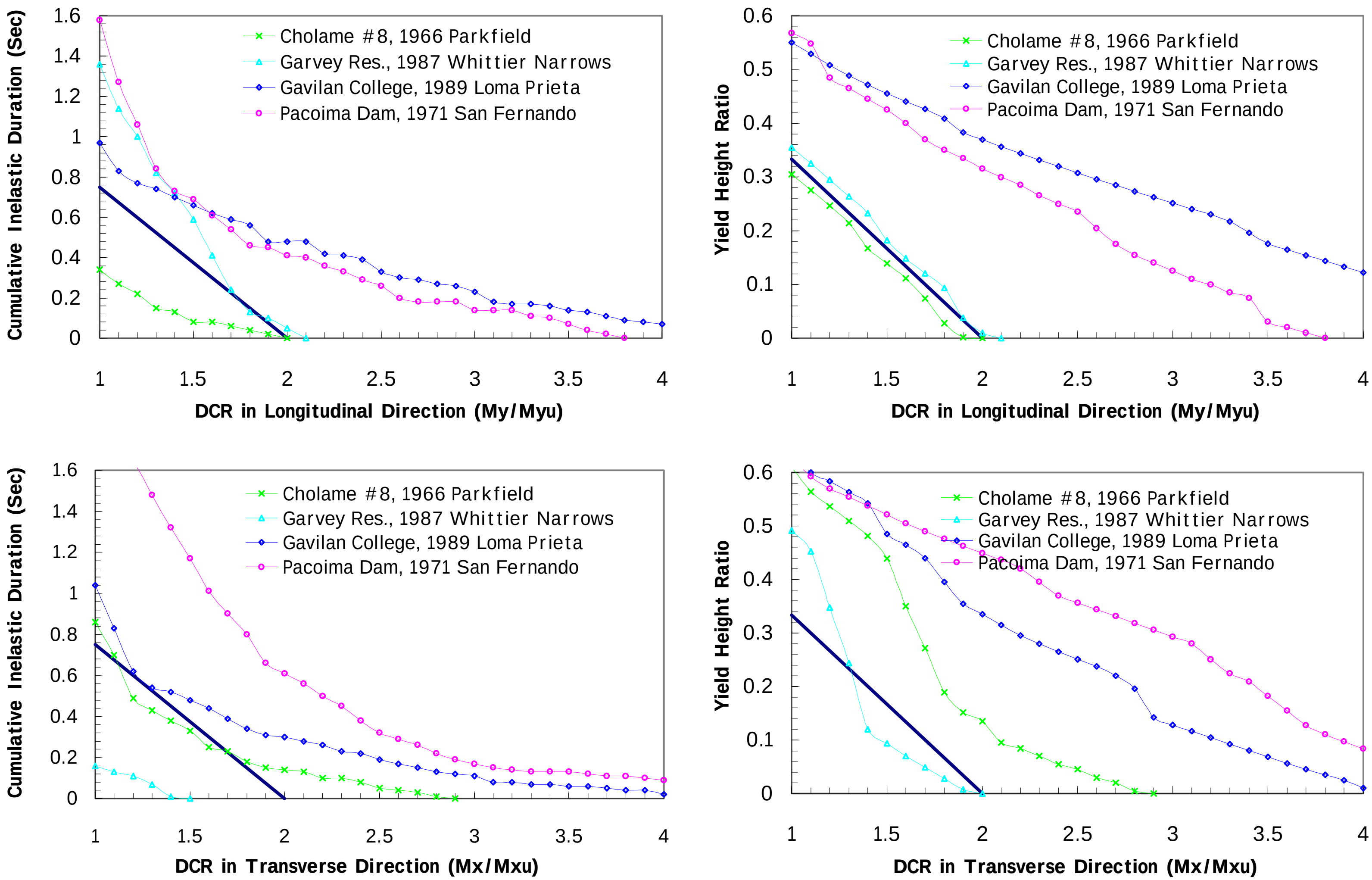




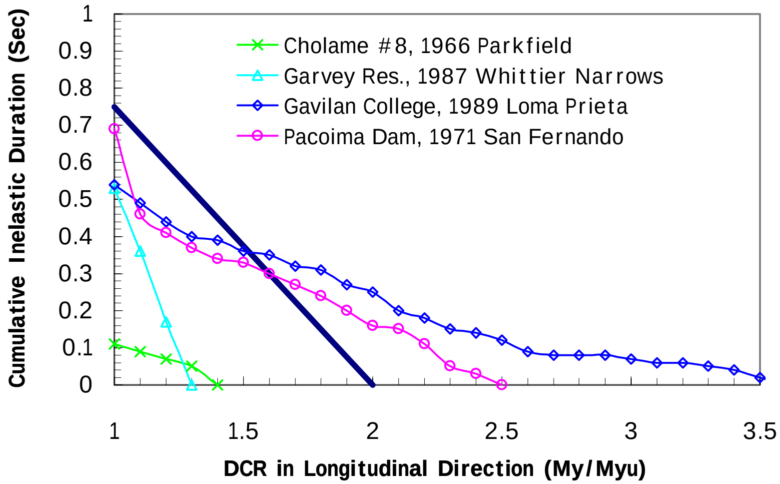


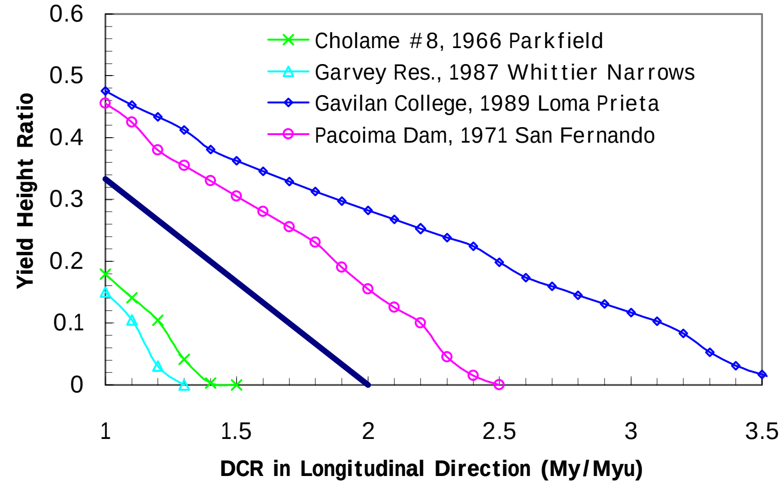


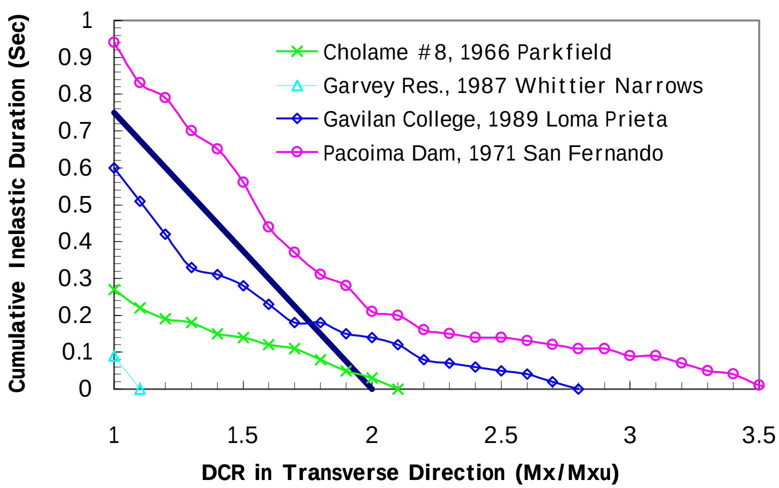
Damage Assessment with 5% damping

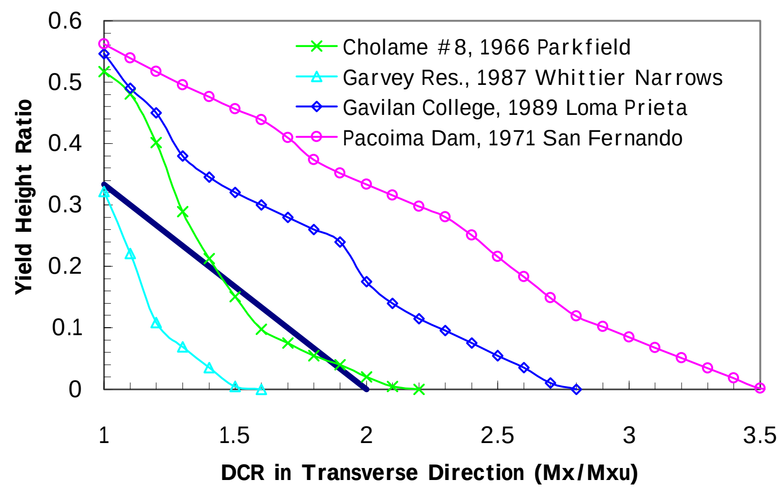


Damage Assessment with 10% Damping







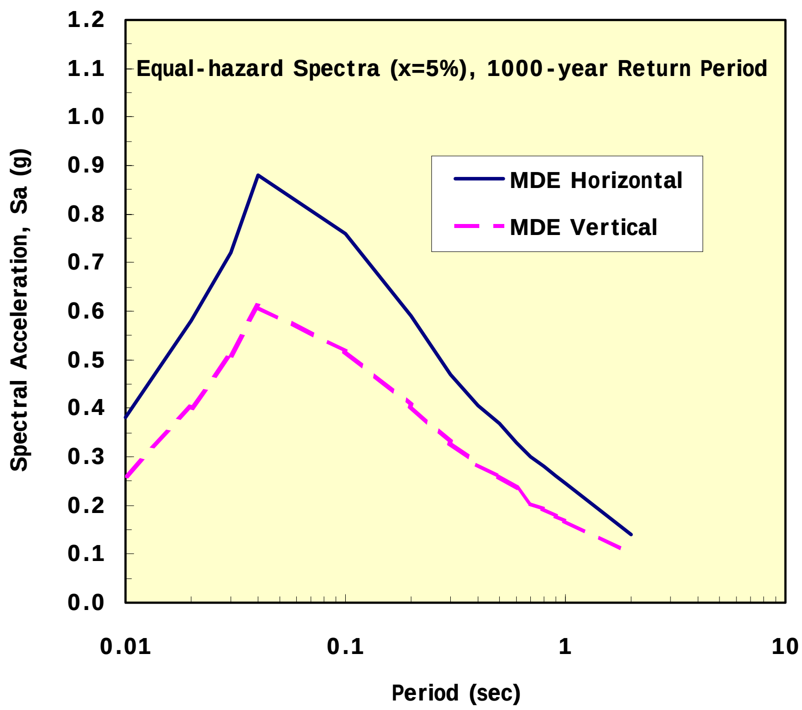


Dynamic Soil-Pile-Structure Interaction Analysis of Olmsted Lock

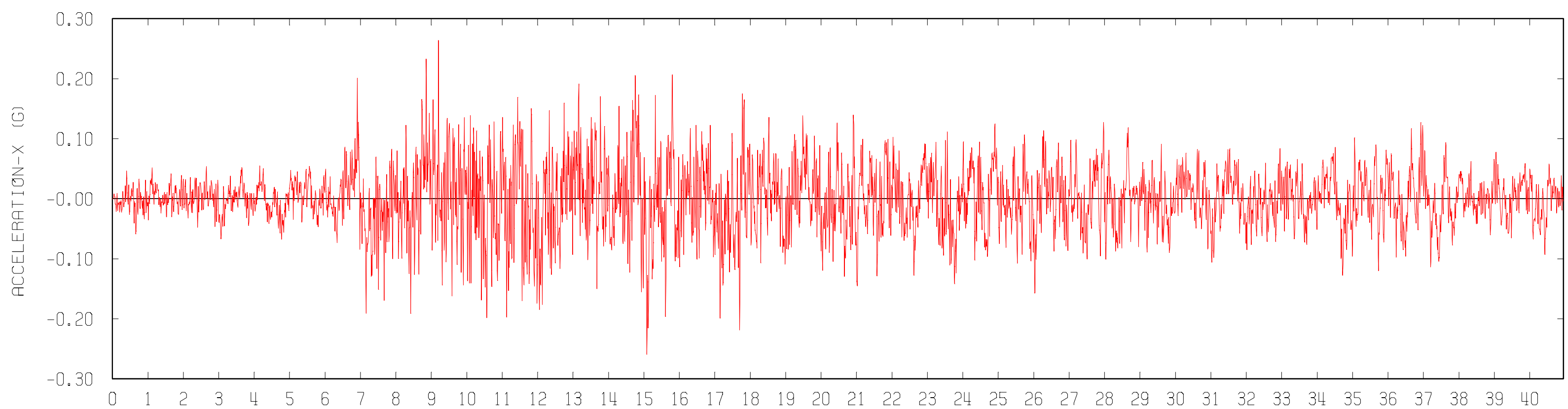
- MDE spectra and ground motion time histories
- Idealization of site soil profiles and estimates of dynamic soil properties
- Development of finite element models of the soil-pile-lock structure system
- Analysis of static loading
- Analysis of dynamic loading
- Results and performance evaluation

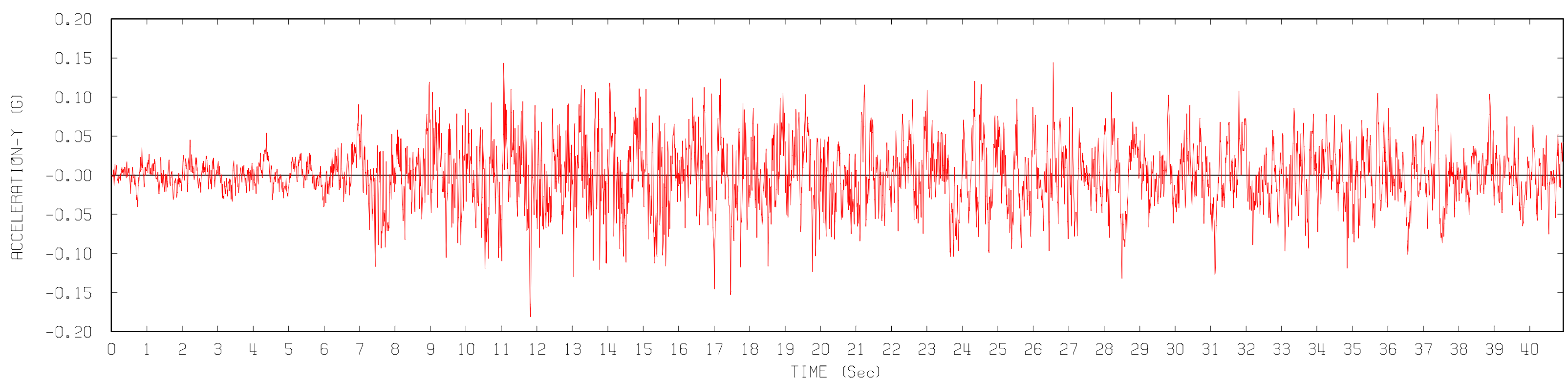
Design Earthquake Motion and Load Combination Cases

Case	Seismic Loads		Static Loads	
	Horizontal Excitation	Vertical Excitation	Bending Moment	Axial Force
1	+	+	+	+
2	+	-	+	+
3	-	+	+	+
4	-	-	+	+

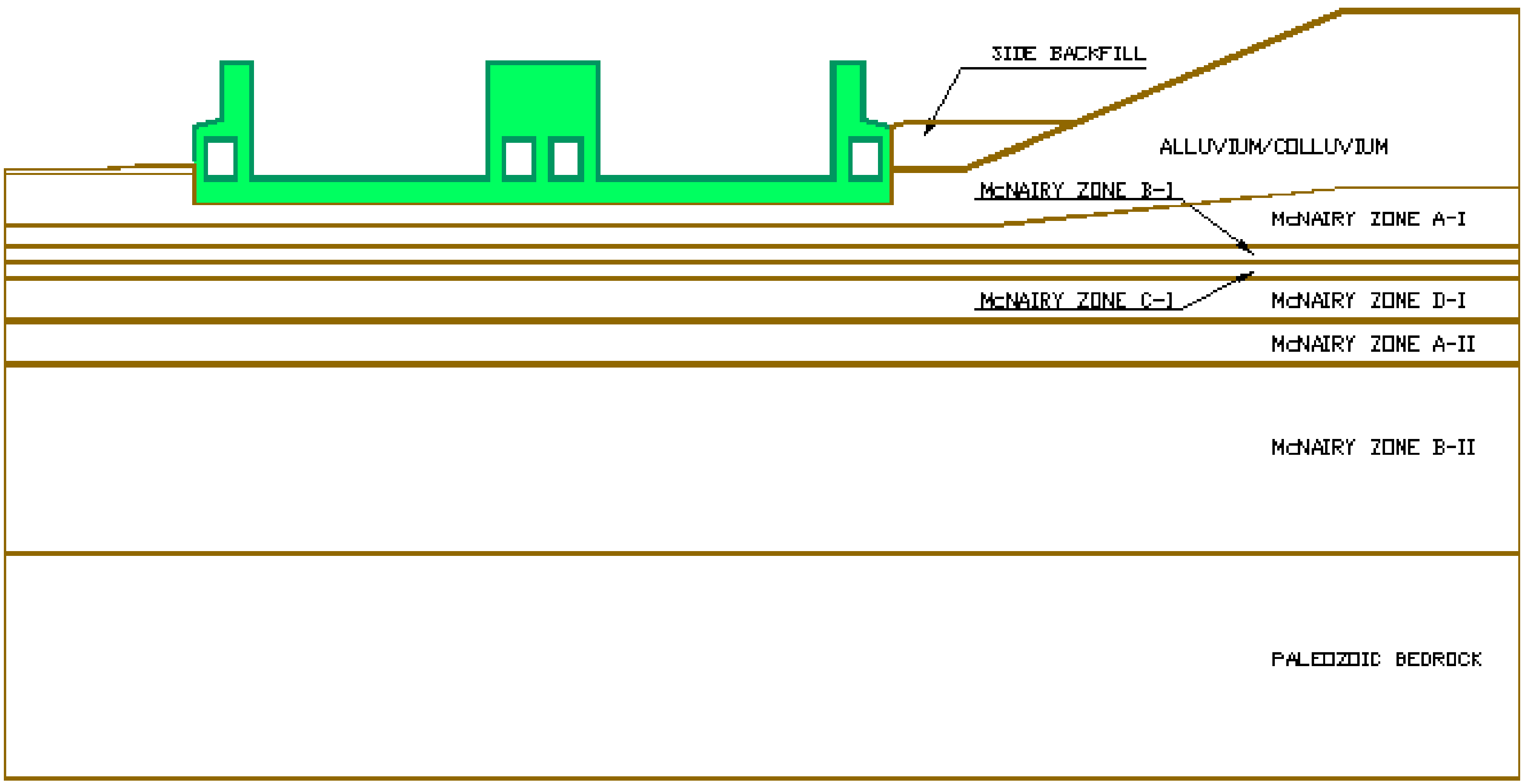


Earthquake Ground Motion Acceleration Time Histories

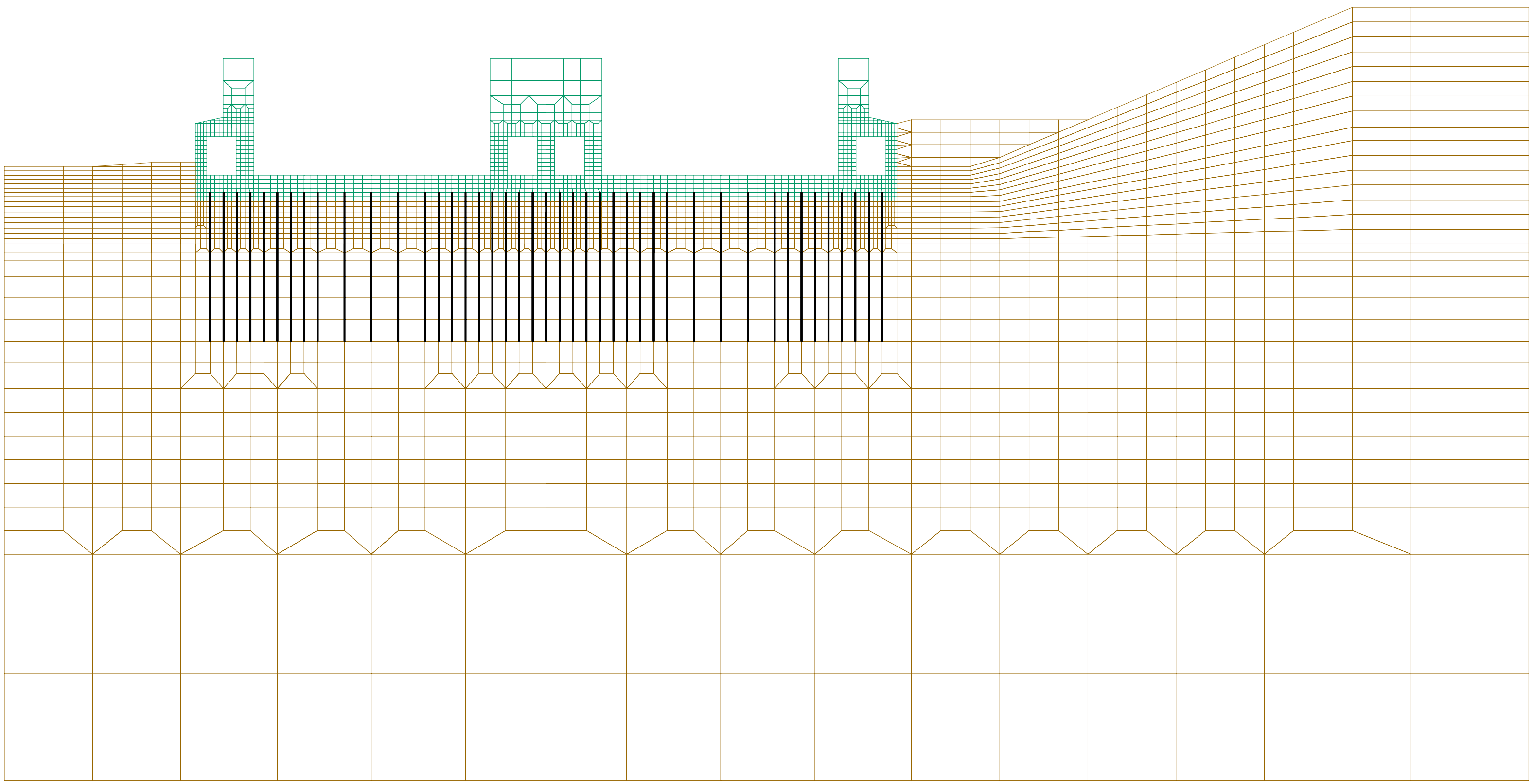




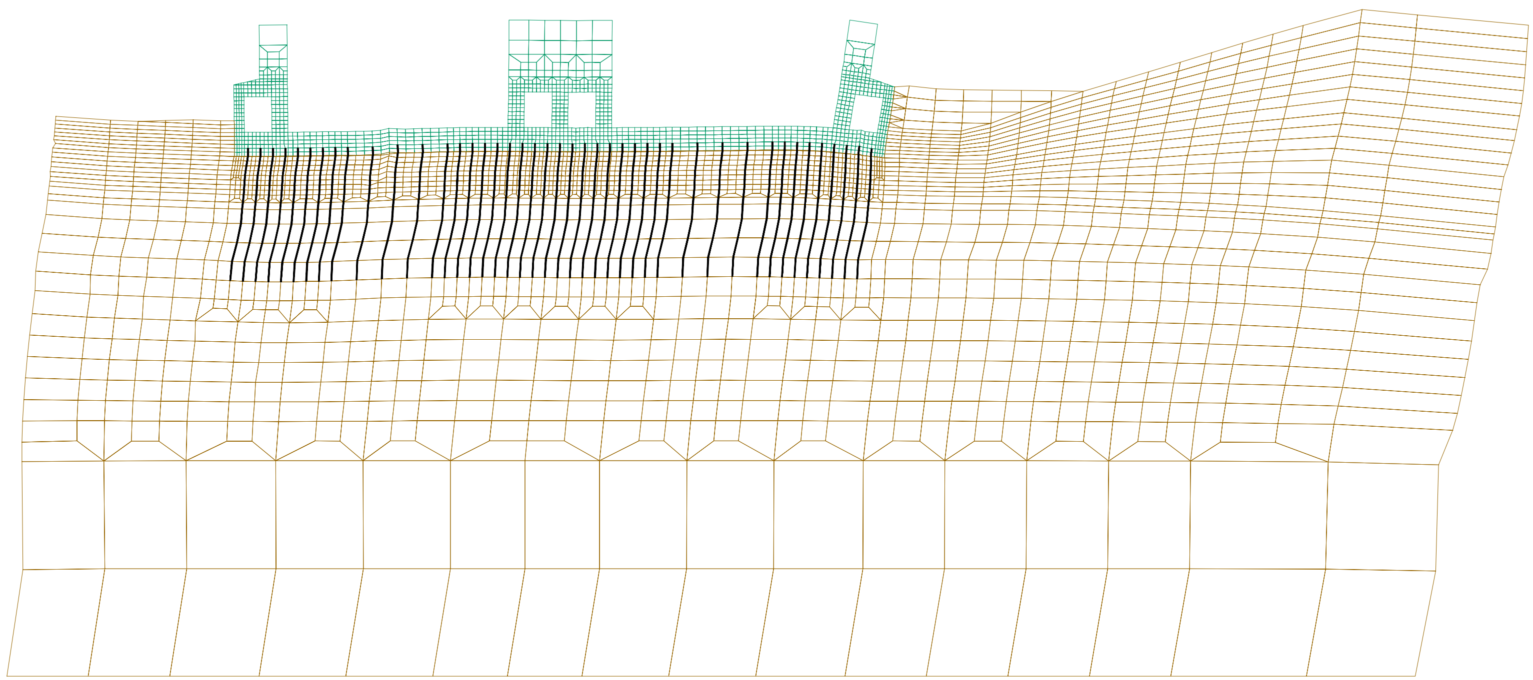
Stratigraphic Profile at Olmsted Site



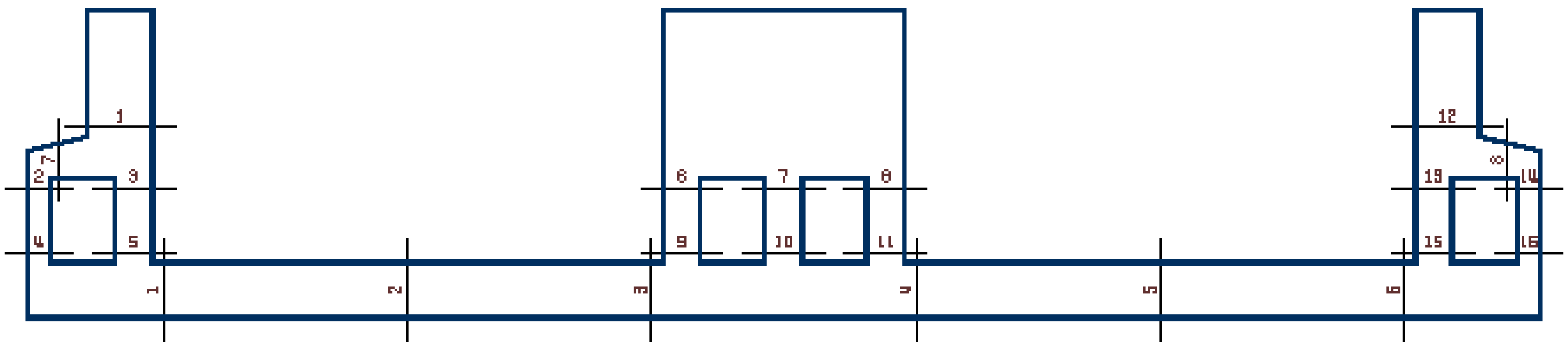
Soil-Pile-Structure Interaction Analysis



Snap Shot of Maximum Deflection of Lock-Pile-Foundation



Evaluation of Lock Section Forces



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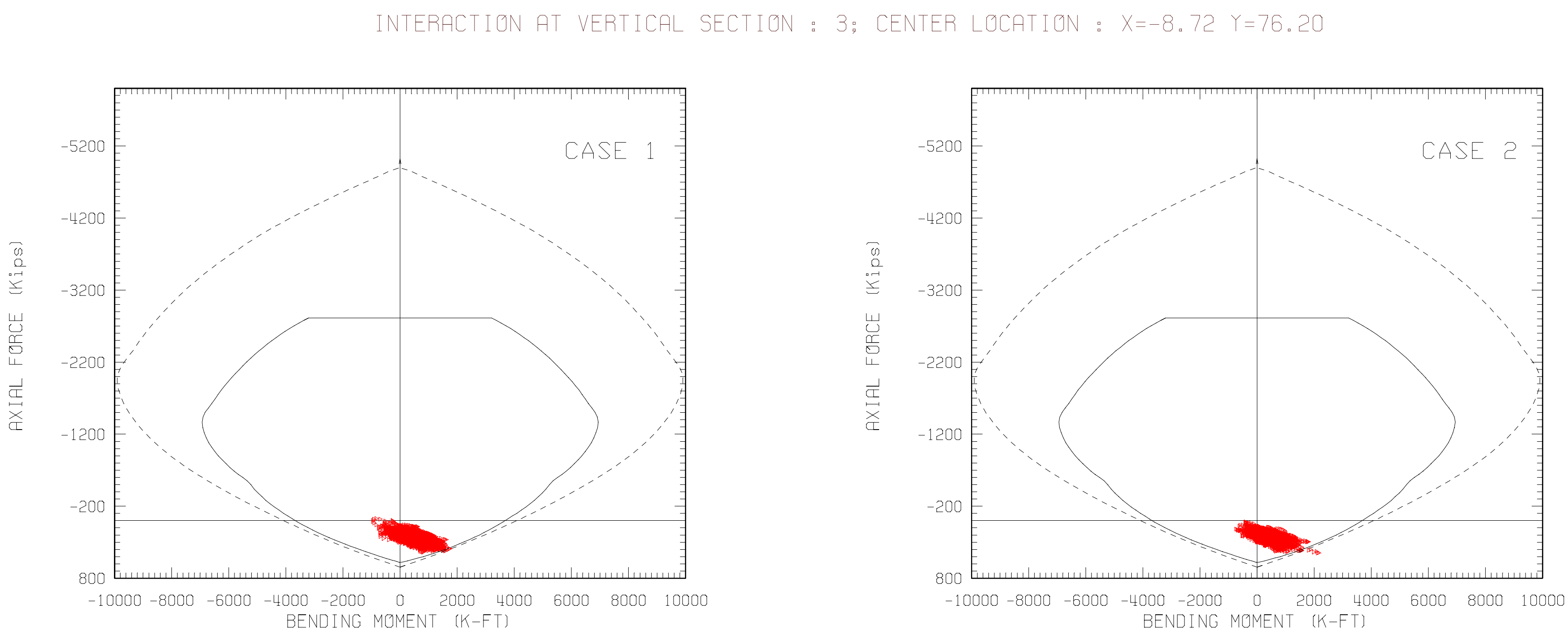
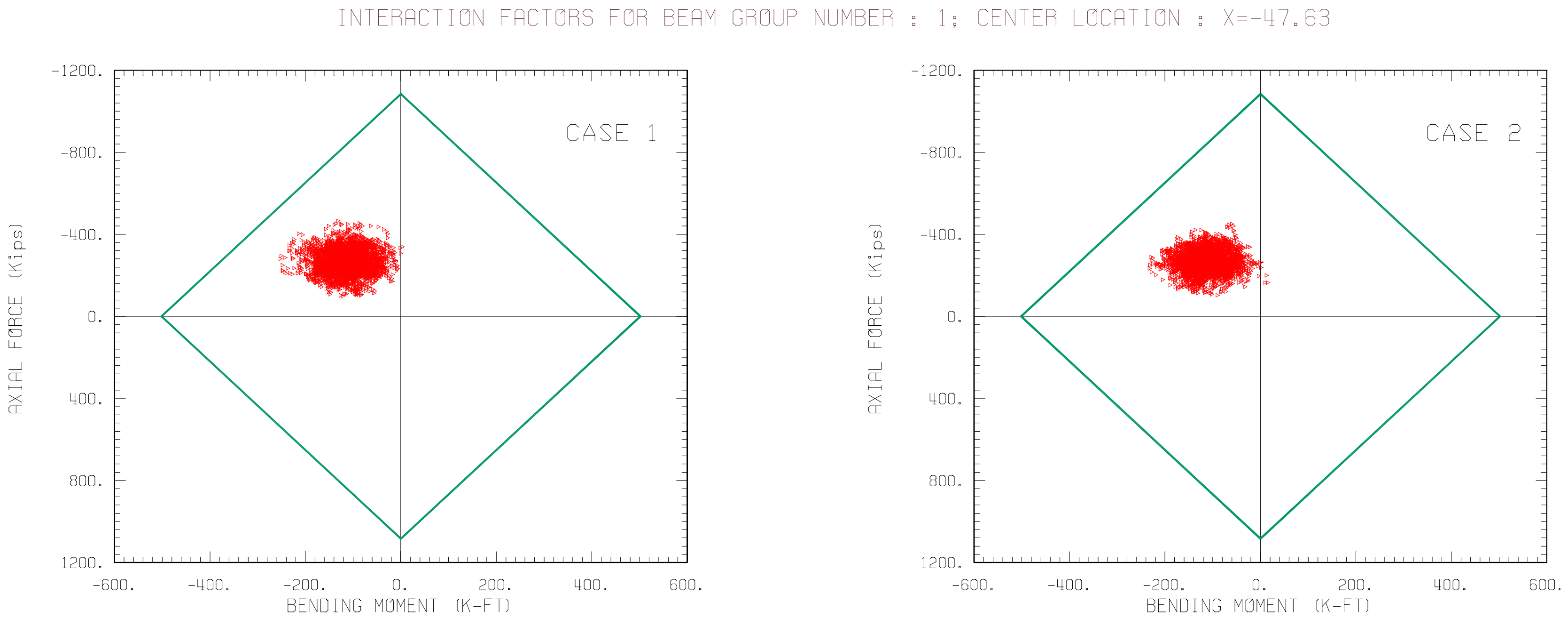
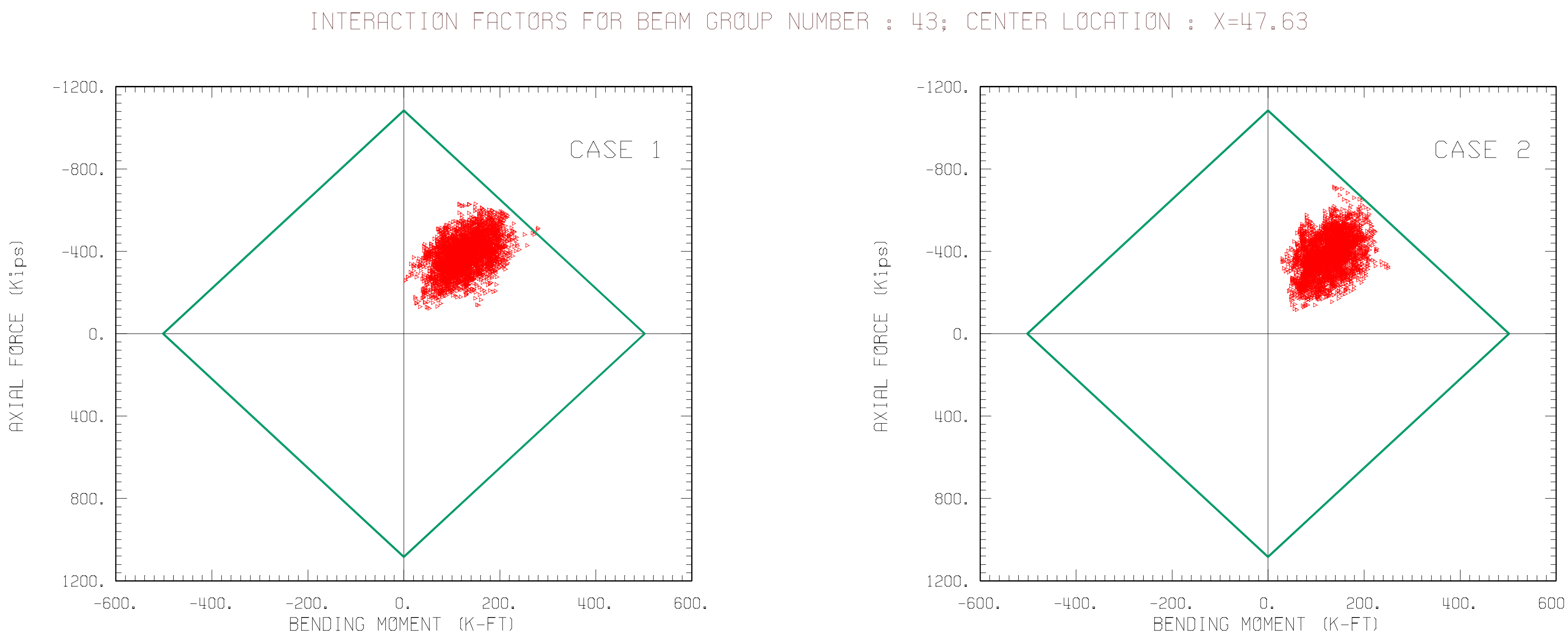


Evaluation of Pile Forces and Moments

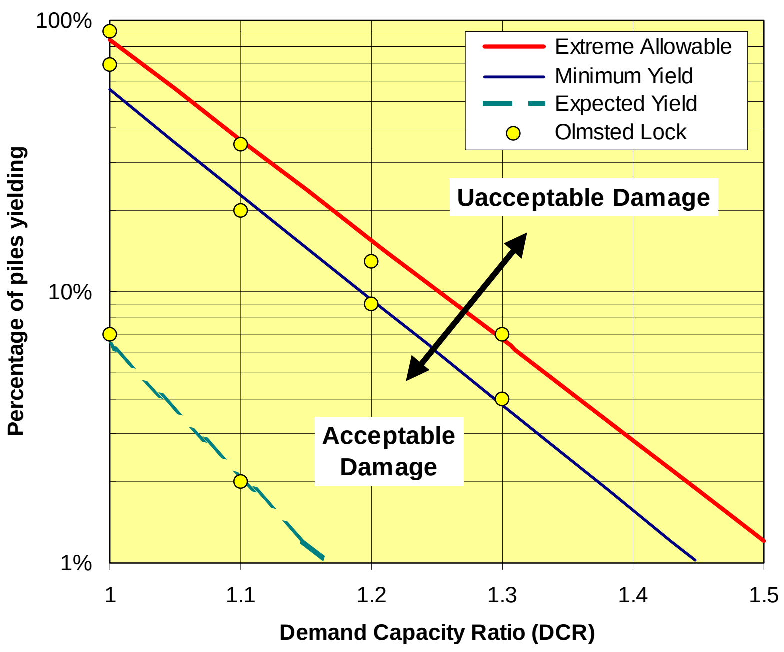


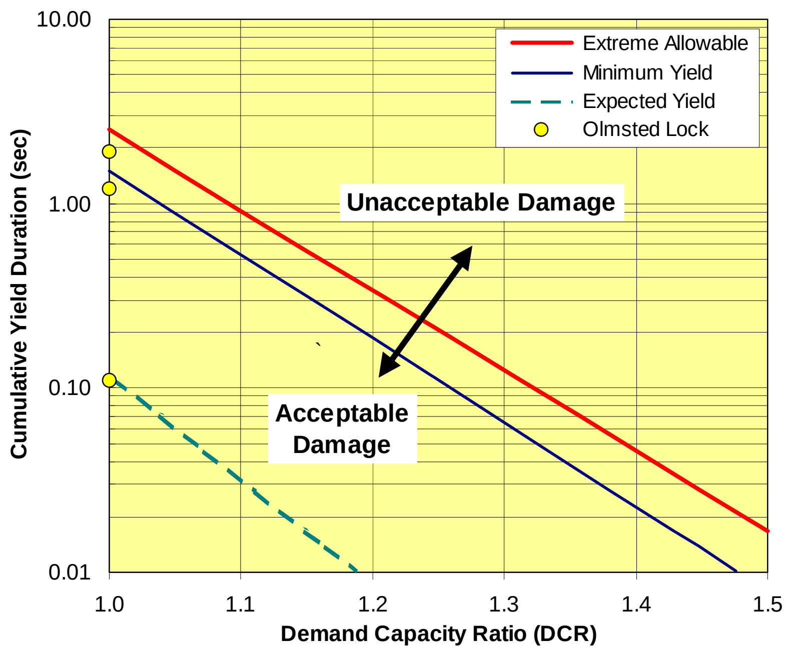
Evaluation of Pile Forces and Moments



Performance Criteria for Lock H-Piles

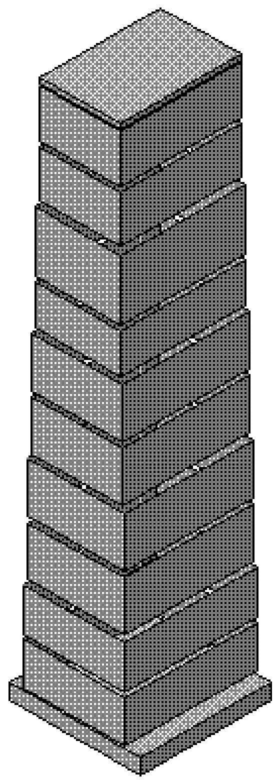
- "*Expected Yield*" Case: yielding should be limited to less than 10% of piles and cumulative yield duration should not exceed 0.1 sec
- DCR of concrete sections should not exceed 1.5 and those exceeding one be limited to less than 10% surface area of the lock.



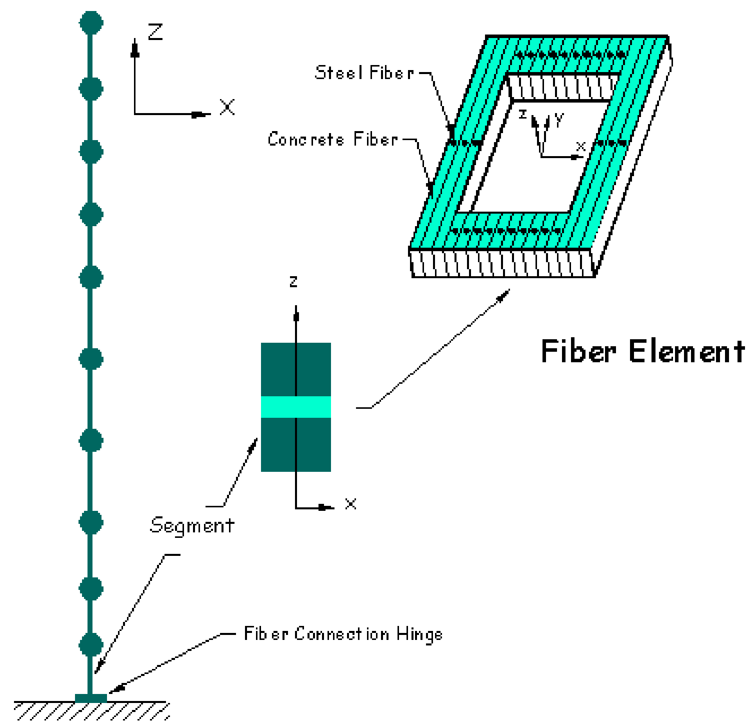


Pushover Analysis of Intake Towers with RC Fiber Element

3D View of Example Tower



Stick Model



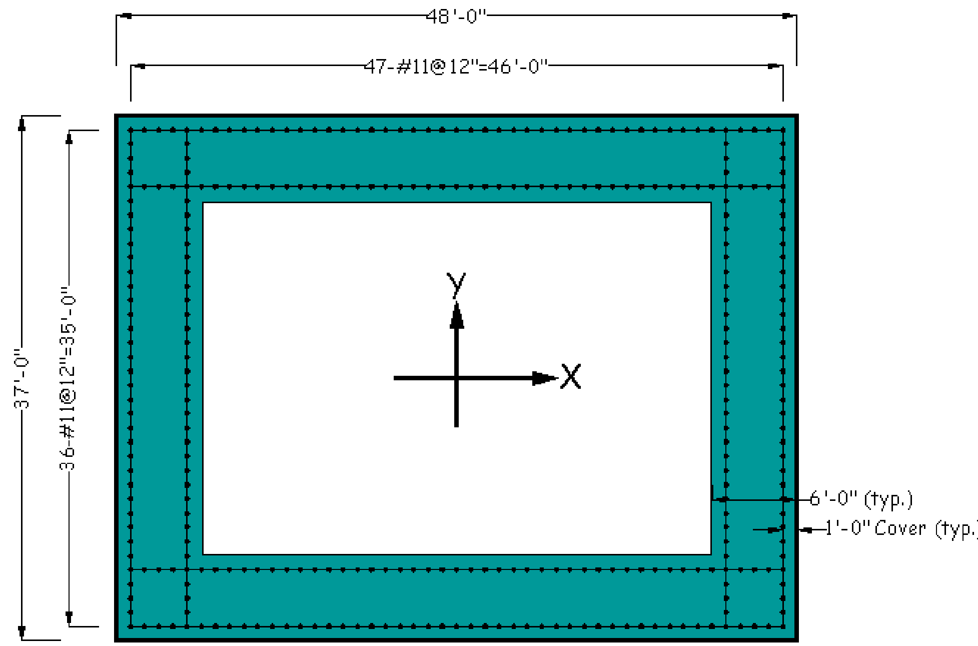
Tower Section Properties and Re-bar Arrangement

Pushing in Longitudinal Direction

Parameter		Value
Width	(b)	37.00 ft
Depth	(h)	48.00 ft
Cross Section Area	(A)	876.00 ft^2
Moment of Inertia	(I_{yy})	243,792 ft^4
Nominal Moment	(Mn_y)	718,814 k-ft
Cracking Moment	(M_{cr})	620,900 k-ft

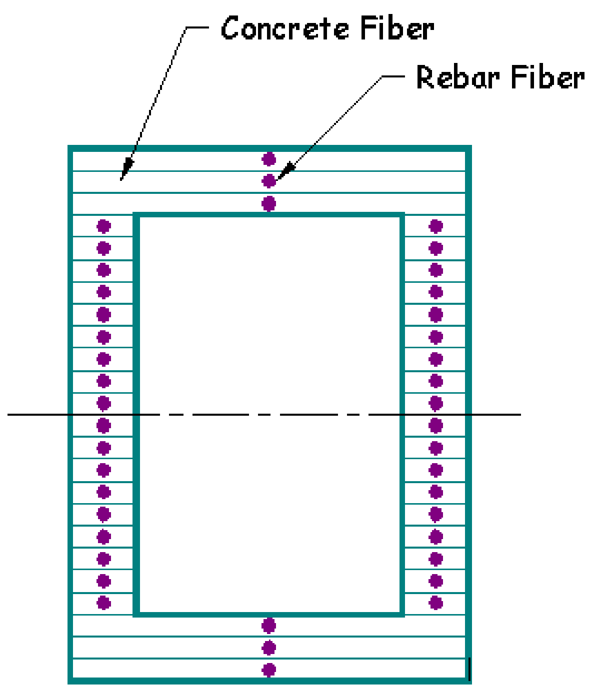
Pushing in Transverse Direction

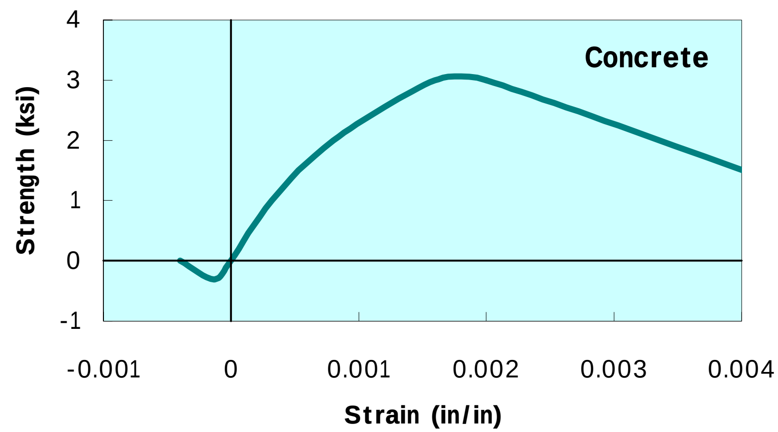
Parameter		Value
Width	(b)	48.00 ft
Depth	(h)	37.00 ft
Cross Section Area	(A)	876.00 ft^2
Moment of Inertia	(I_{xx})	155,737 ft^4
Nominal Moment	(Mn_x)	518,879 k-ft
Cracking Moment	(M_{cr})	507,900 k-ft

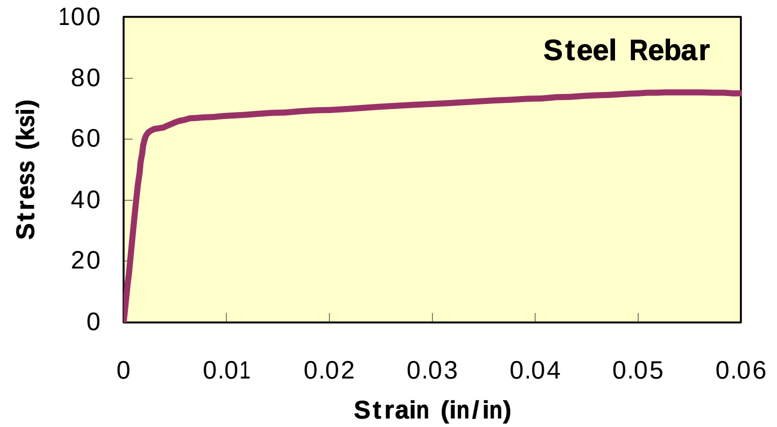


Nonlinear RC Fiber Element

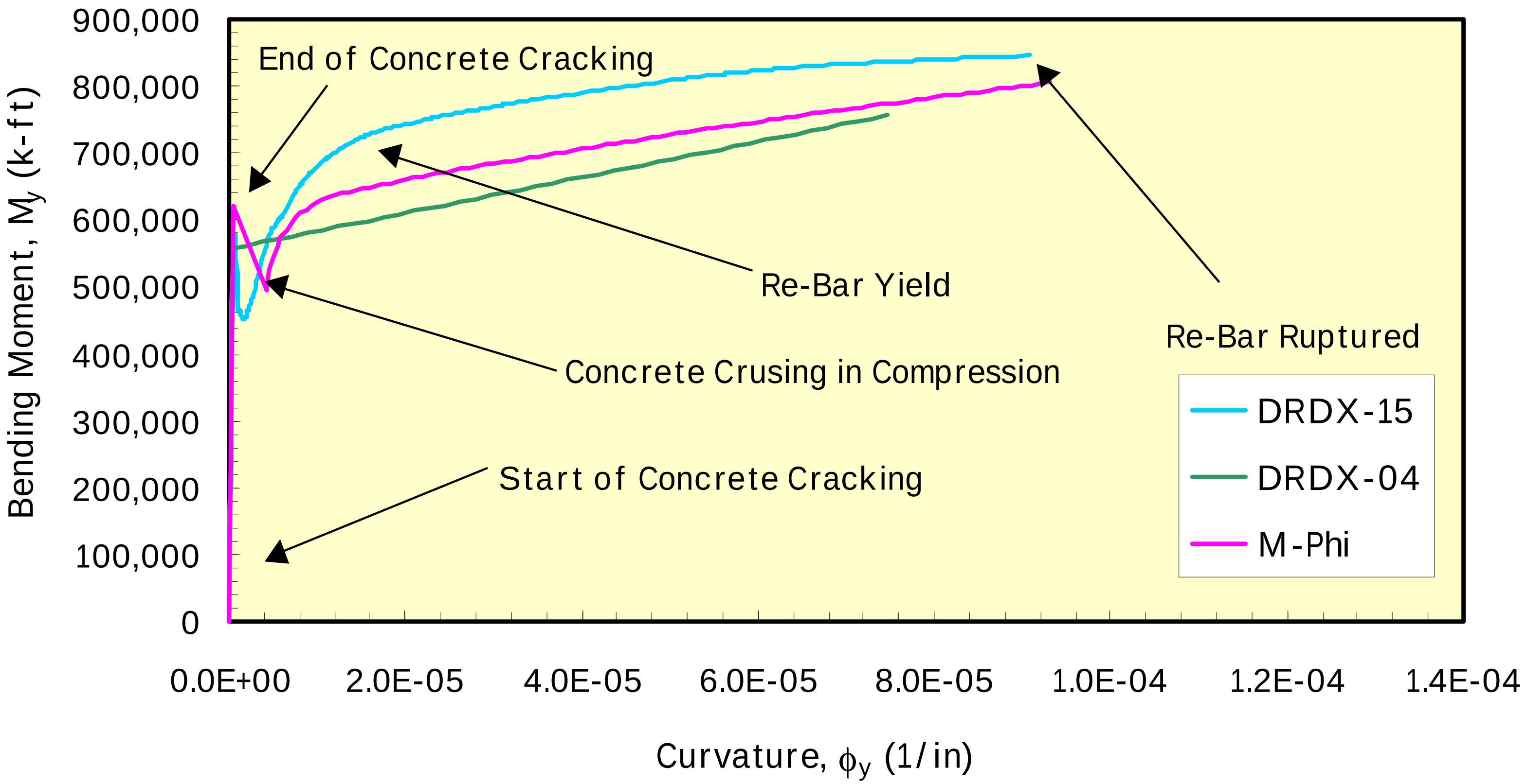
- RC Fiber Element



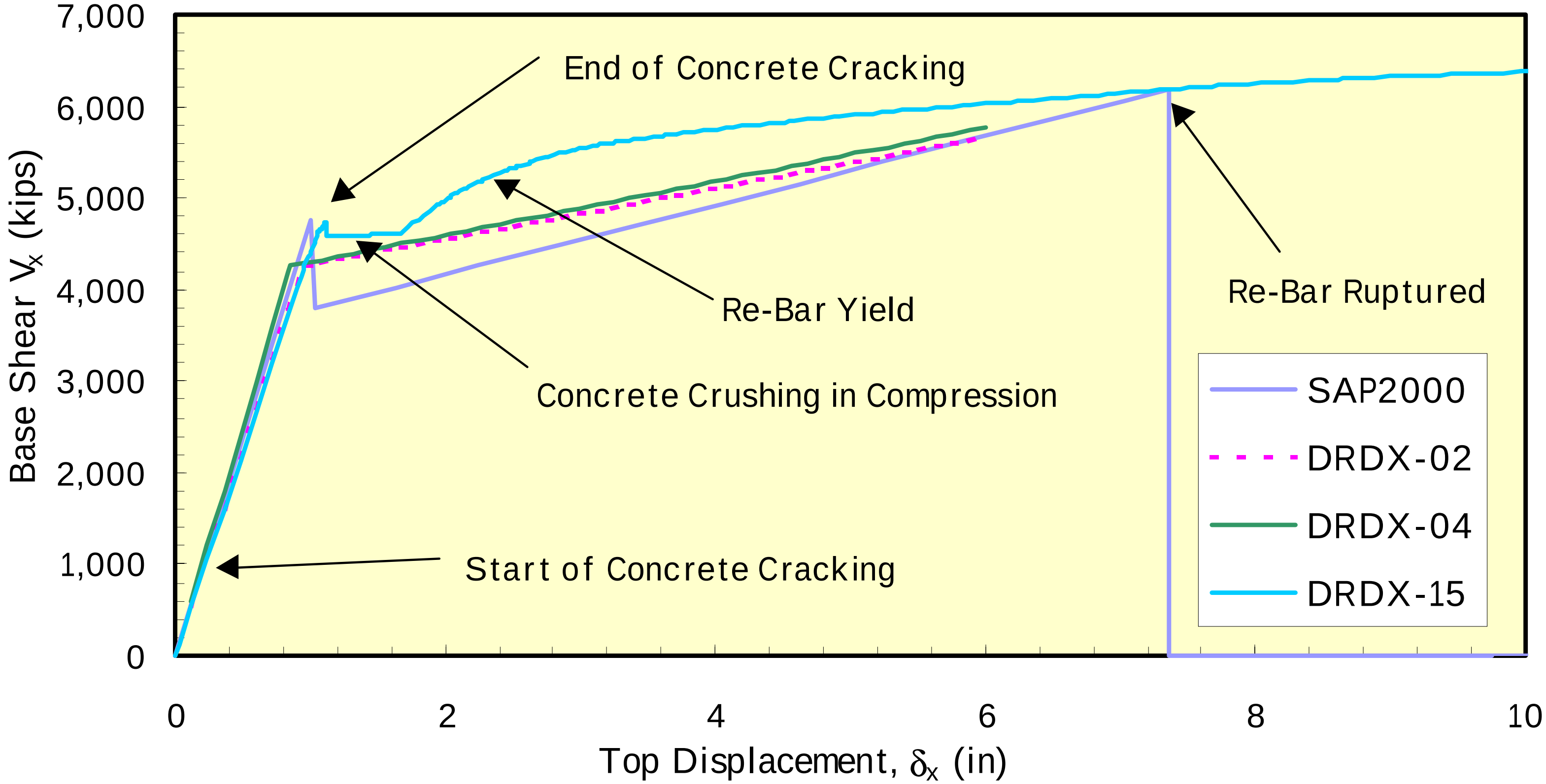




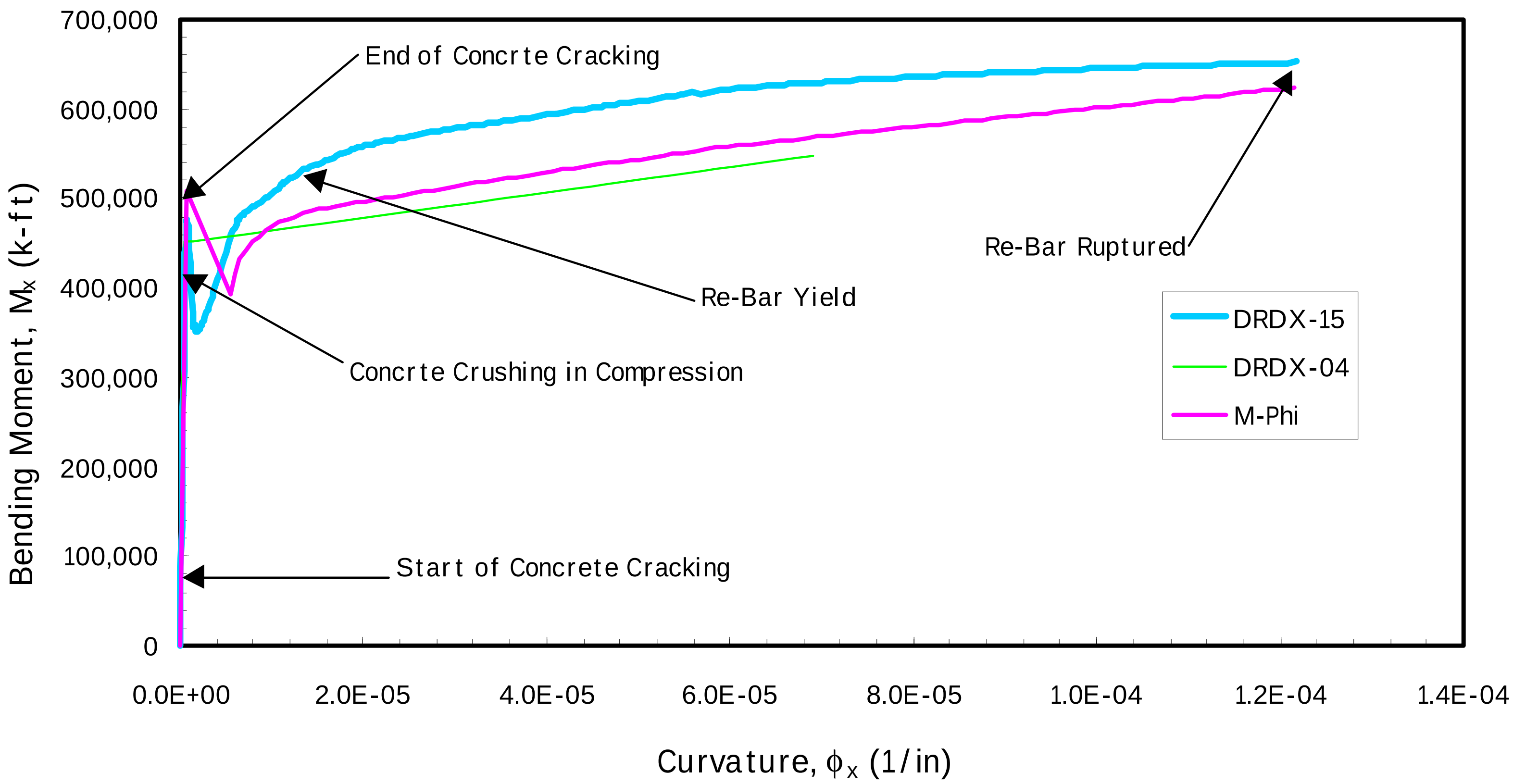
Moment-curvature Relationships for Example Intake Tower (x-dir)



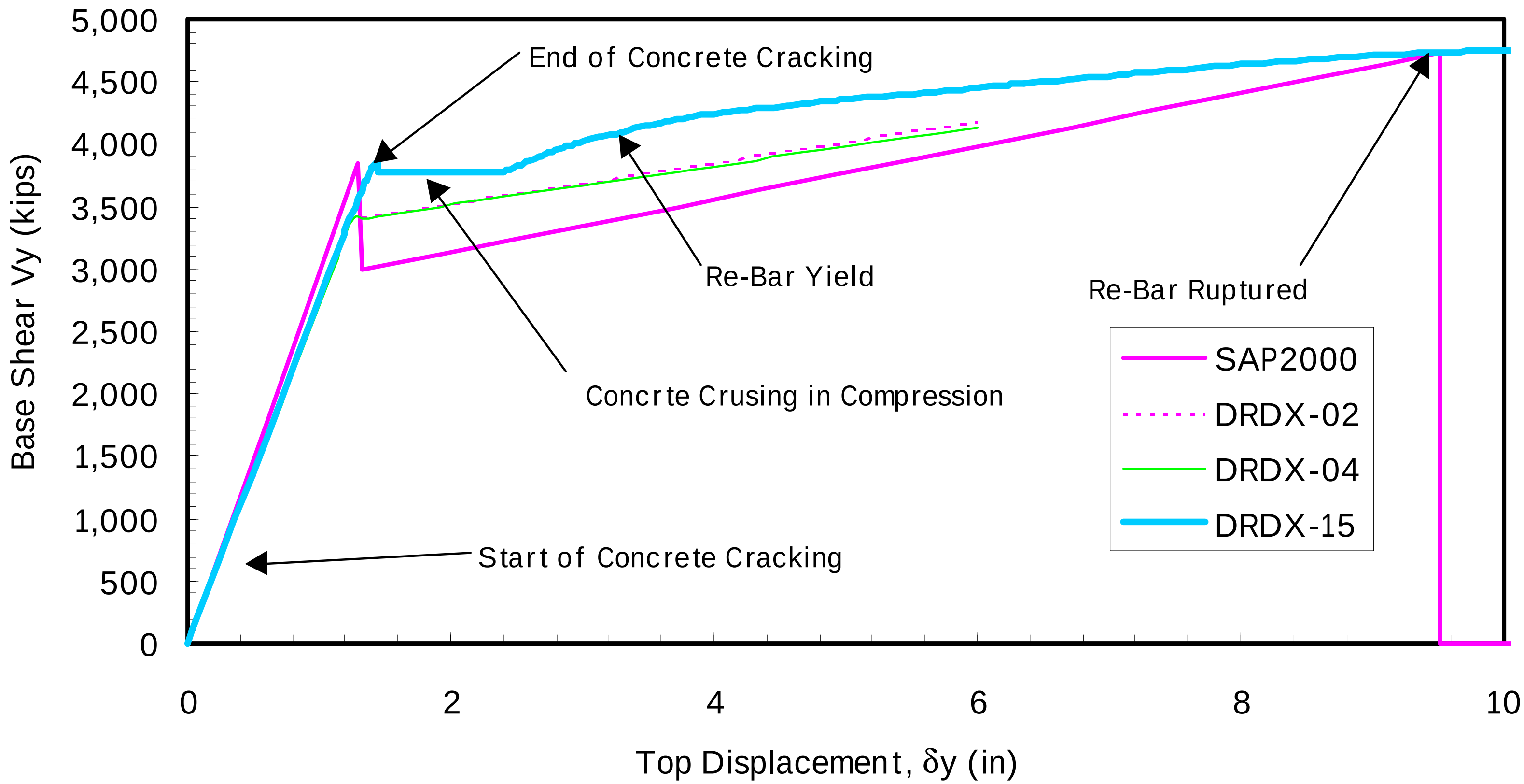
Pushover Curves for Example Intake Tower (x-dir)



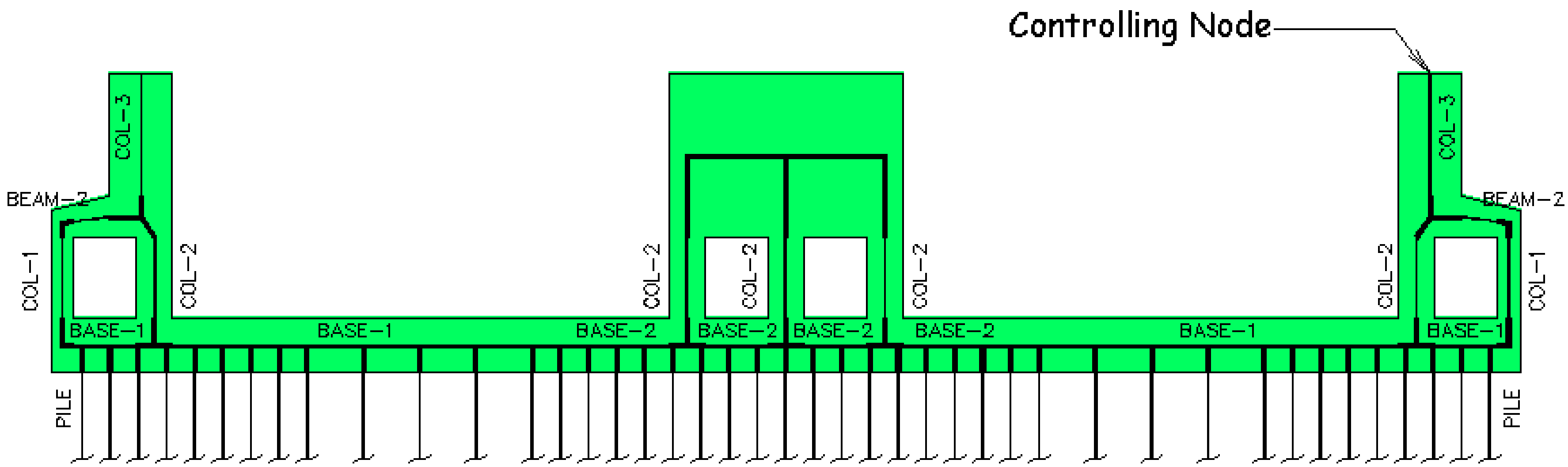
Moment-curvature Relationships for Example Intake Tower (y-dir)



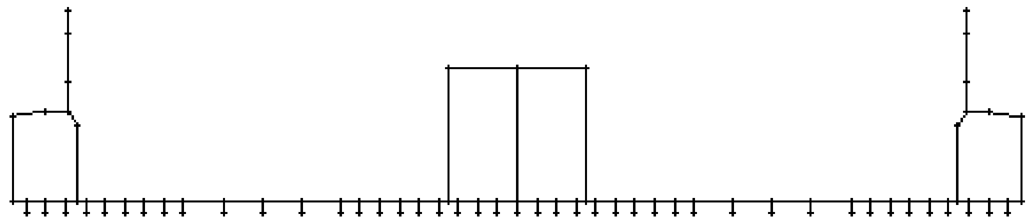
Pushover Curves for Example Intake Tower (y-dir)



Pushover Analysis of Navigation Locks

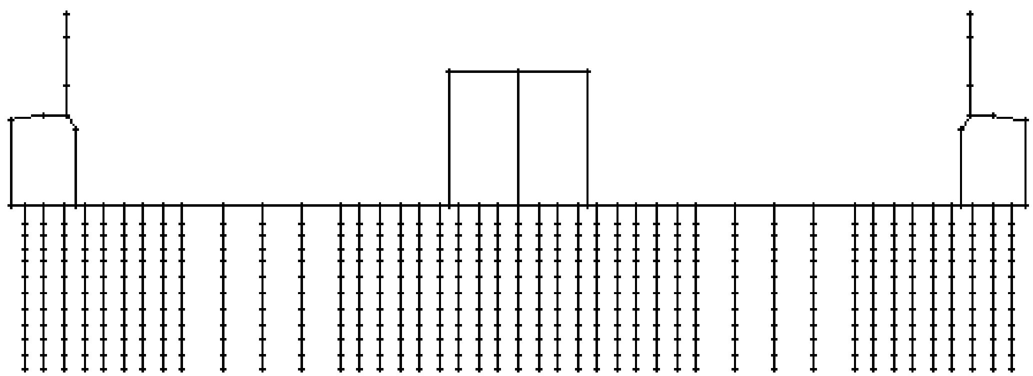


Nonlinear FE Model for Pushover Analysis of Navigation Locks



- <u>Lumped Model</u>

  Pile foundation represented by lumped springs



- <u>Full Model</u>

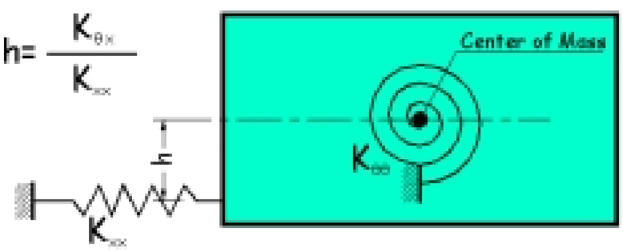
  Pile foundation represented by nonlinear beam-column and soil springs (p-y curves)

Nonlinear Pile Foundation Models

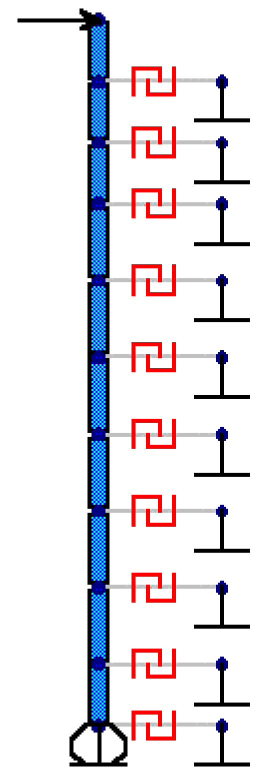
- Nonlinear Lumped Pile-Head Springs

$$\begin{Bmatrix} V_x \\ M_y \end{Bmatrix} = \begin{bmatrix} K_{xx} & K_{x\theta} \\ K_{\theta x} & K_{\theta\theta} \end{bmatrix} \times \begin{Bmatrix} u_x \\ \theta_y \end{Bmatrix}$$

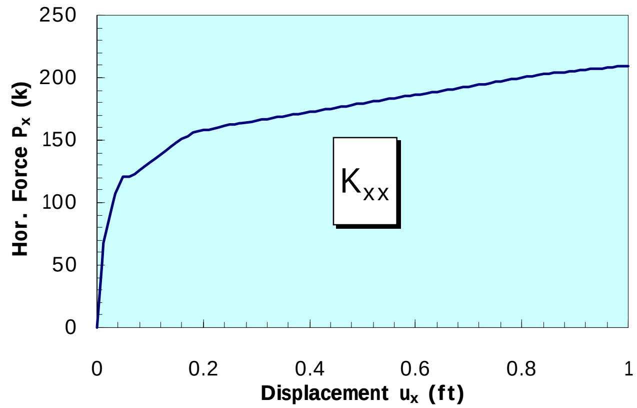
$$h = \frac{K_{\theta x}}{K_{xx}}$$

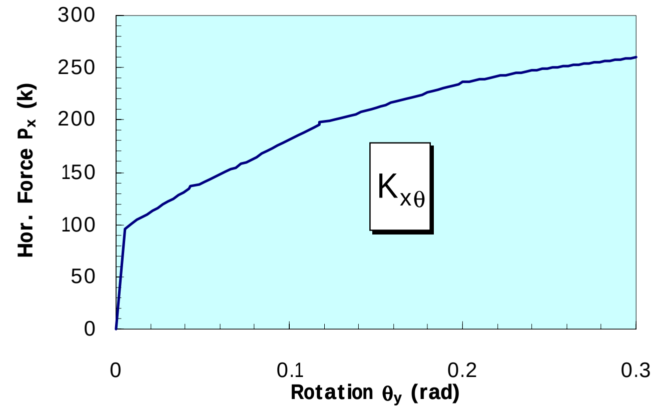


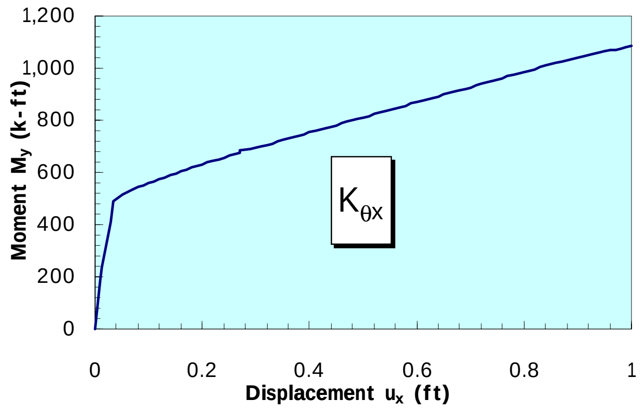
- Nonlinear Pile-Soil Model

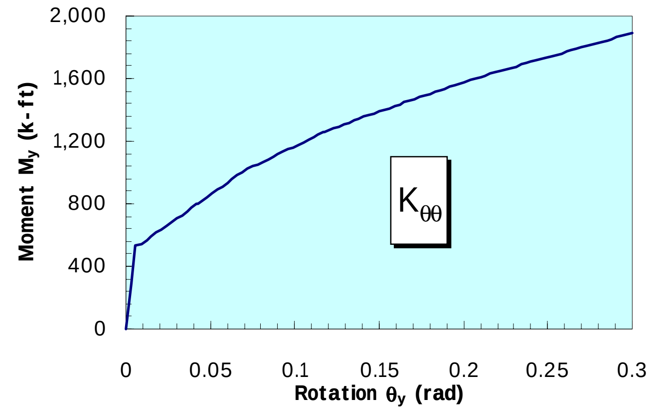


Nonlinear Pile-head Stiffness



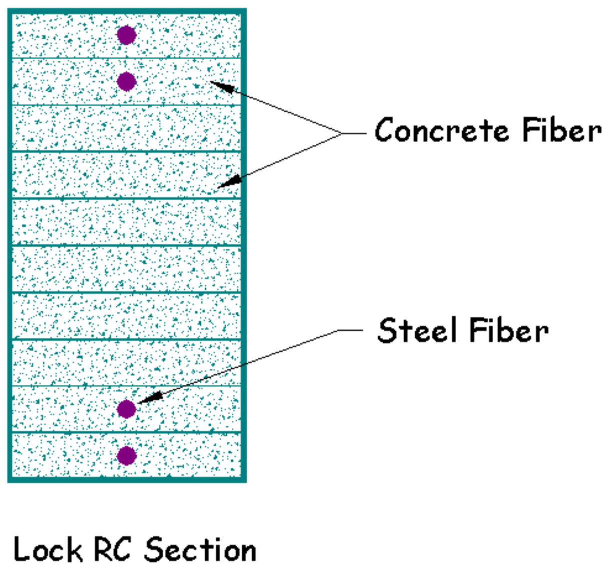


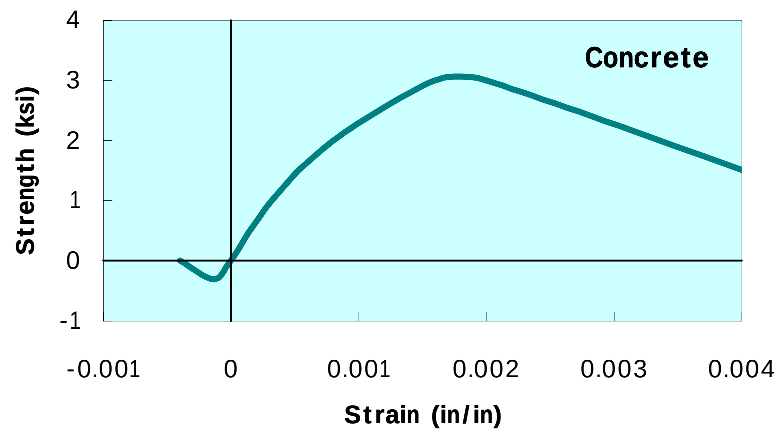


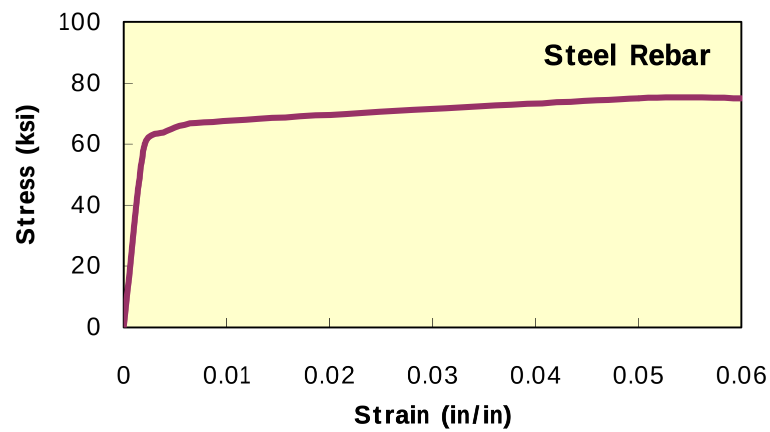


Nonlinear Fiber Element

- RC Fiber Element

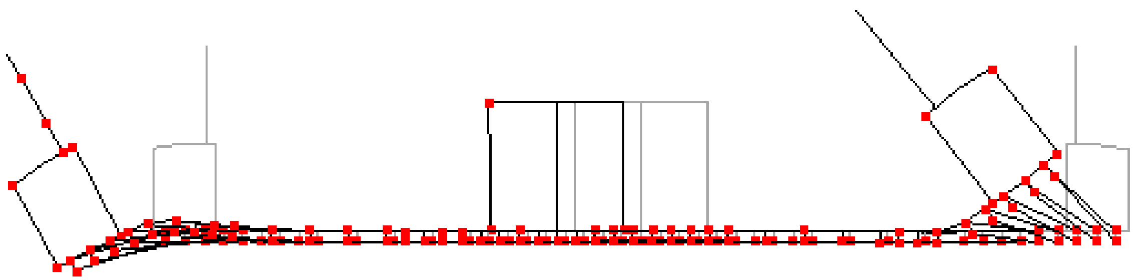




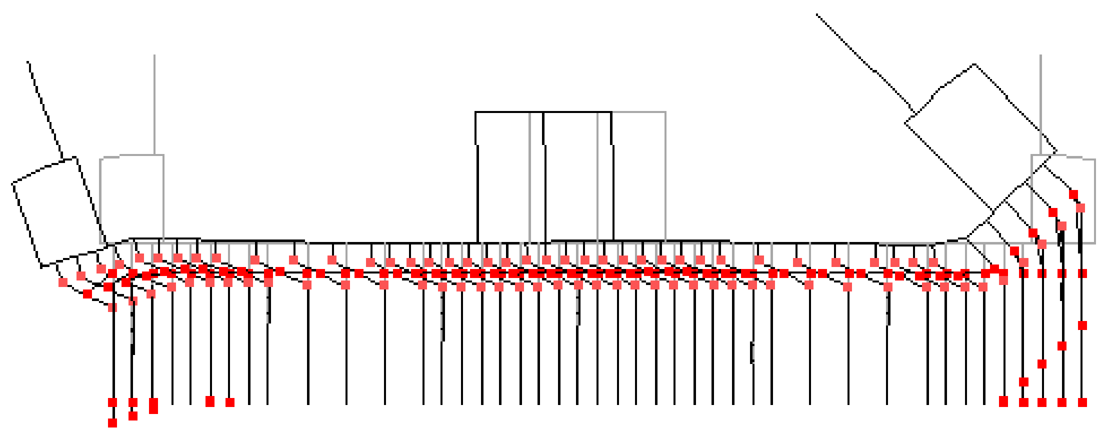


Pushover Deflected Shapes for Pile-Founded Lock

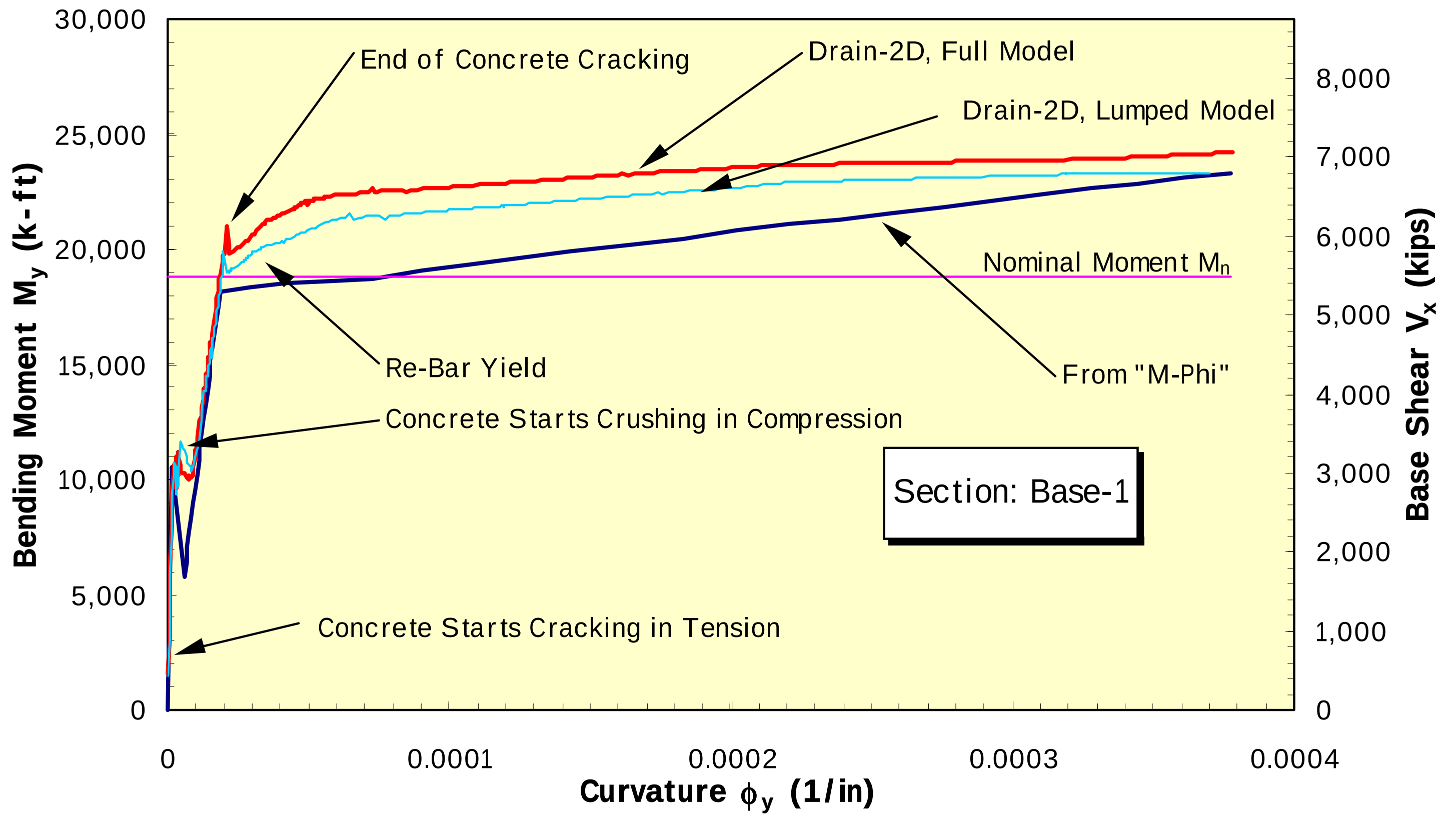
Lumped Model



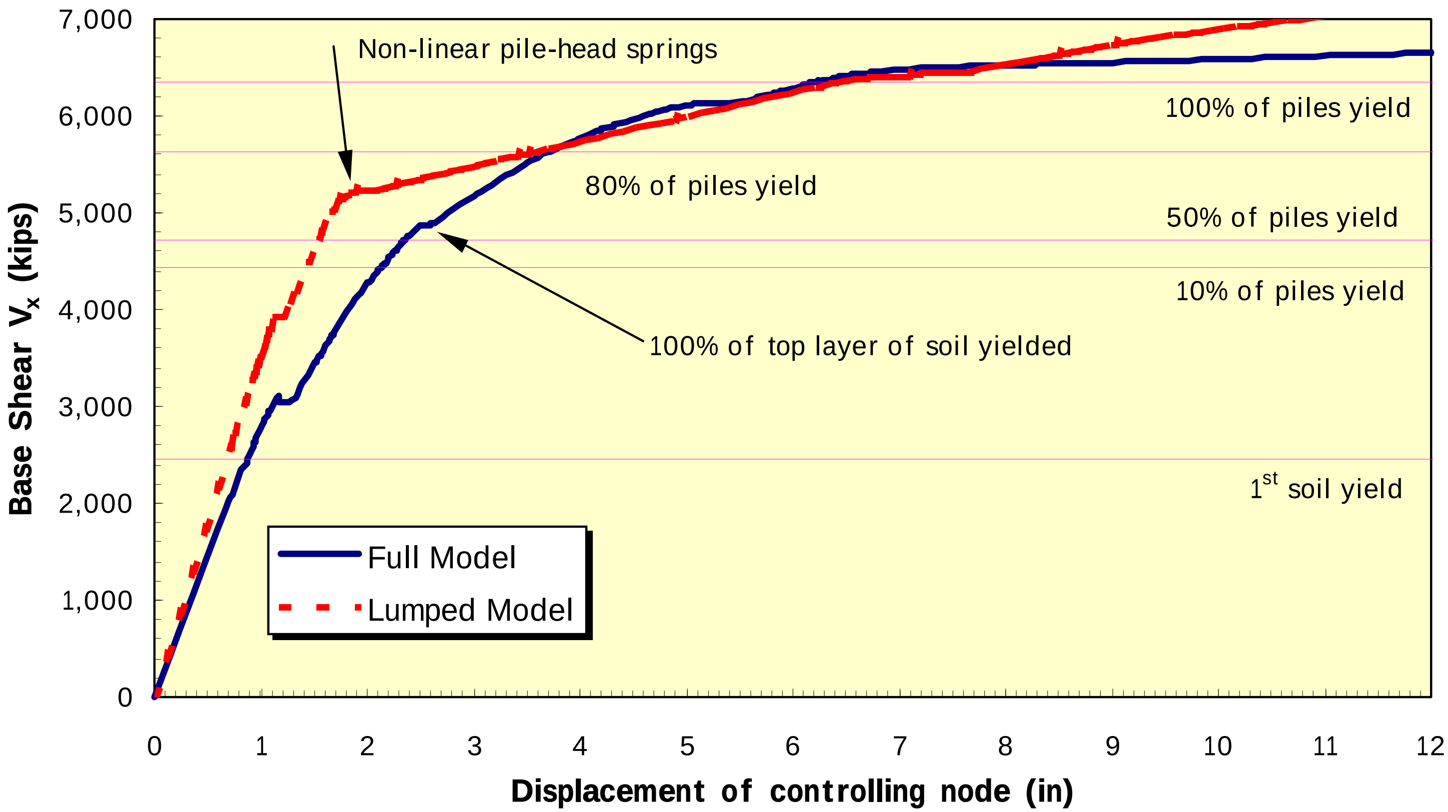
Full Model



Moment-Curvature relationship for Lock Base Slab



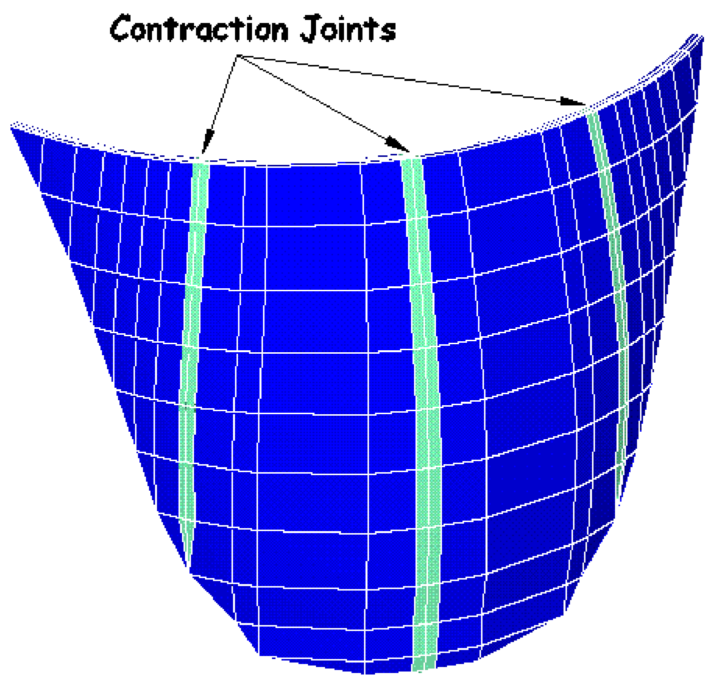
Pushover Curve for Pile-Founded Lock



Nonlinear Analysis of Arch Dams with Joint Opening

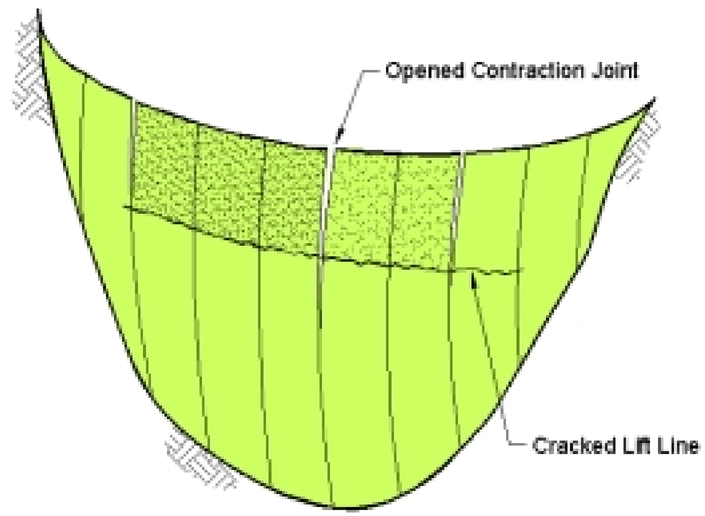
Parameters of Greatest Significance

- Input Earthquake Acceleration Histories
- Joint Strength and Stiffness Properties
- Frictional Resistance of Joints
- Number of Joints
- Location of Joints

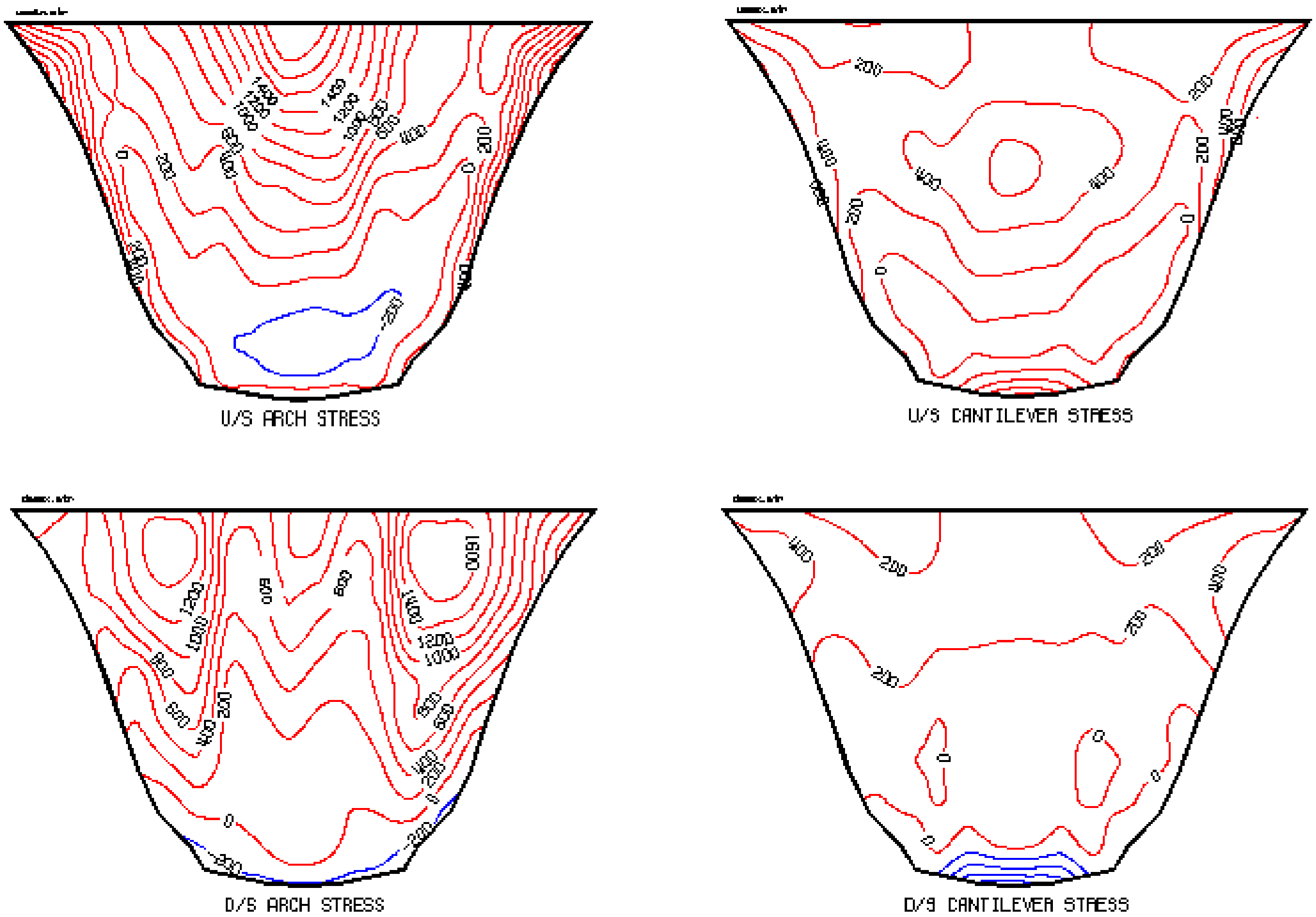


Results and Performance Evaluation

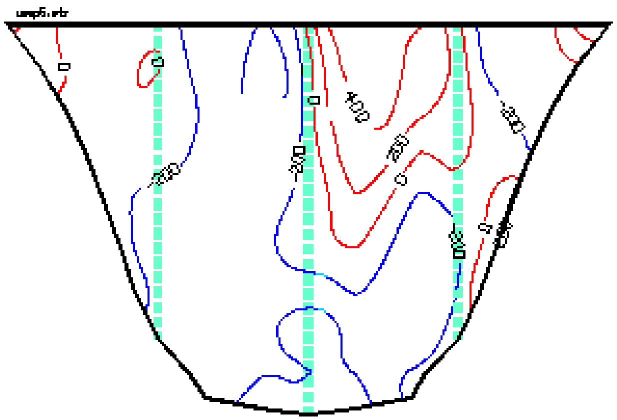
- Envelope of Arch & Cantilever Stress Contours
- Instantaneous Arch & Cantilever Stress Contours
- Extend/History of Contraction Joint Opening/Sliding
- Extent/History of Lift Joint Cracking/Opening/Sliding
- Understanding of Dam Behavior & Potential Failure Modes, if Severe Joint Opening/Cracking



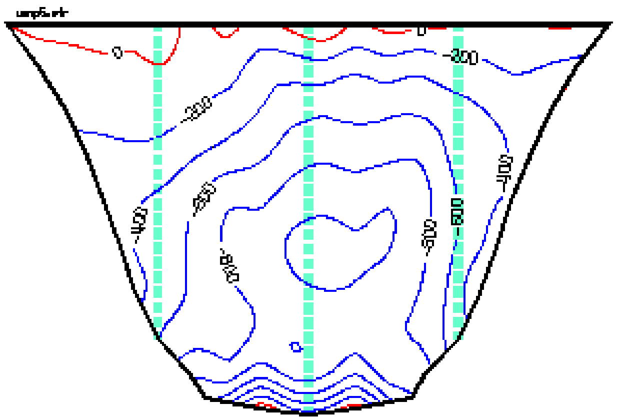
Envelope of Maximum Stresses (monolithic dam)



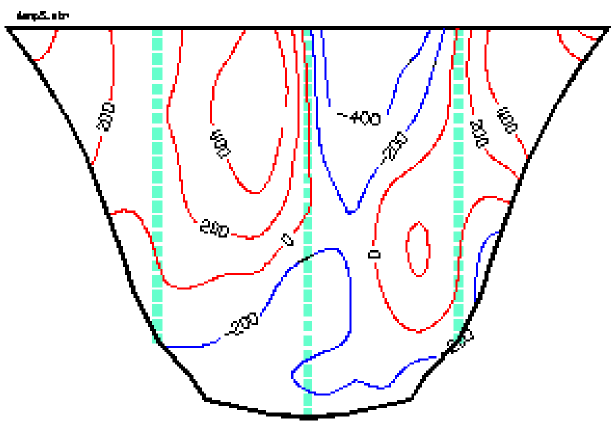
Concurrent Maximum Stresses (joints permitted to open)



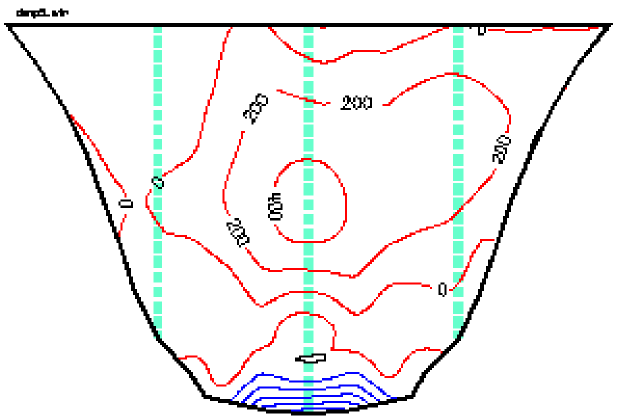
U/S ARCH STRESS



U/S CANTILEVER STRESS

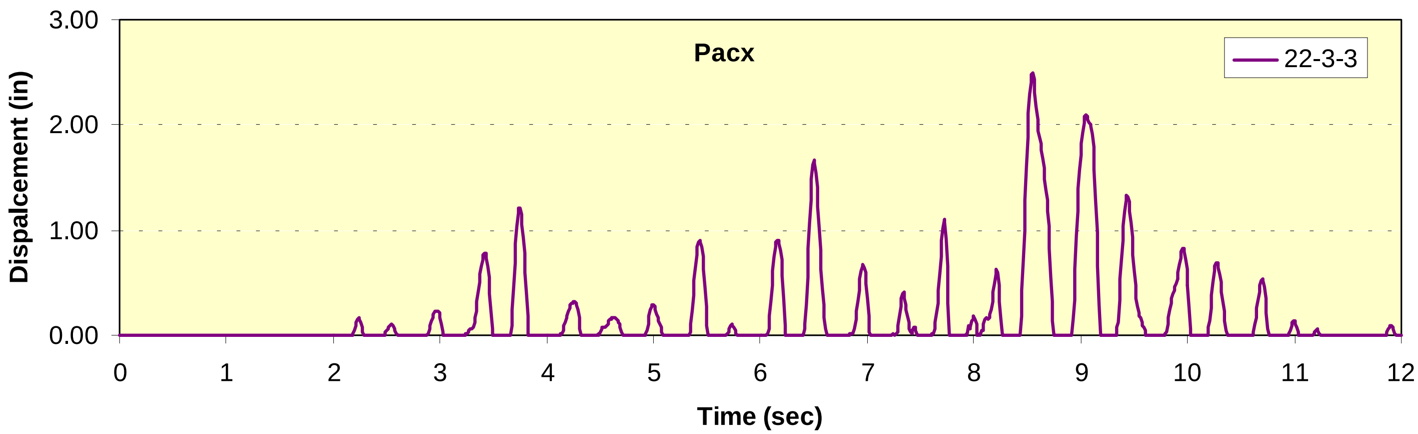


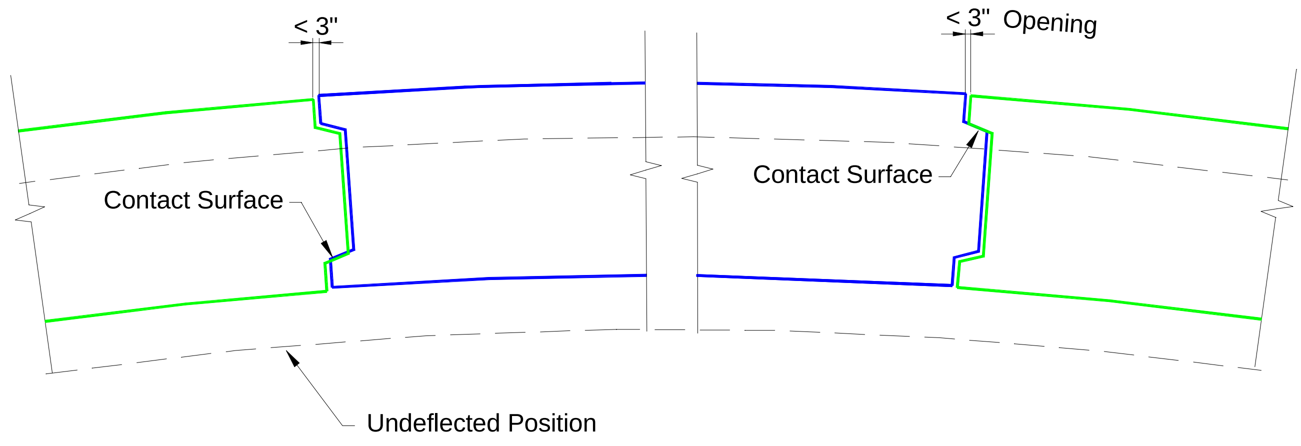
D/S ARCH STRESS



D/S CANTILEVER STRESS

Time History of Maximum Joint Opening





Historic Performance of Dams

- <u>Shih-Kang Gravity Dam</u>:

  Incident:

  The Chi-Chi, Taiwan Earthquake of Sep. 21, 1999

Fault rupture was most dramatic at Shih-Kang Dam. It passed directly beneath the right end of the dam and caused severe damage. The offsets were roughly 10m vertical and 2 m horizontal. Prior to this earthquake, the Chelungpu fault was not mapped at this site.

Typical Seismic Deficiencies and Corrective Measures

- Bartlett Multiple Arch Dam: 287’ high (Phoenix, CA)
  - Deficiency: The upper portions of arches would be overstressed under an MCE event and might fail

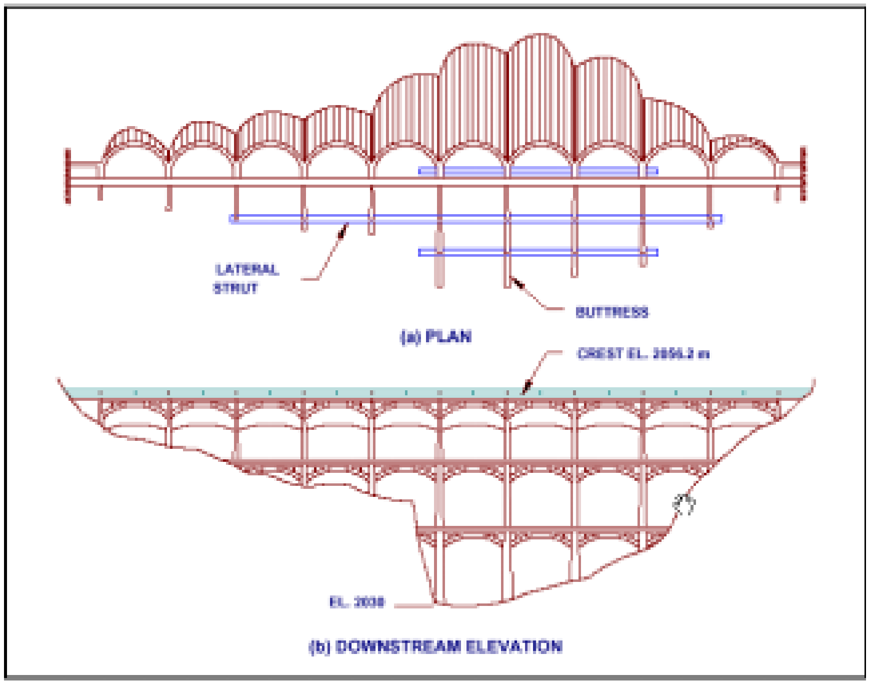
- Clear Creek Dam: 83’ high thin arch dam (Yakima, WA)
  - Deficiency: Unstable under MCE
  - Modification: Was converted from a thin arch to a gravity dam by constructing a mass concrete buttress on the

Typical Seismic Deficiencies and Corrective Measures

- Bear Valley Multiple Arch Dam: 80' high, w/ 10 arches (Redlands, CA)

  Deficiency:
  - Did not meet seismic safety requirements

  Modification:
  - Existing arch barrels were filled by mass concrete to strengthen the dam and thus improve its earthquake resistance



Typical Seismic Deficiencies and Corrective Measures

- <u>Gibraltar Dam</u>: 195'-high arch dam (Santa Barbara, CA)

  Deficiency:
  - Did not meet seismic safety requirements

  Modification:
  - Roller-compacted concrete buttress was constructed against downstream slope of the dam to improve its earthquake resistance

Typical Seismic Deficiencies and Corrective Measures

- Mathis Dam: 108'-high buttress dam (Tallulah Falls, GA)

  Deficiency:
  - Potential sliding failure under PMF

  Modification:
  - Concrete thrust blocks and tendon anchors were added to improve stability

- Shepaug Dam: 140'-high Concrete Gravity (Southbury & Newton, Connecticut)

  Deficiency:
  - Unstable under new PMF

  Modification:
  - Approximately 100 post-tensioned anchors installed in the dam to improve stability

Typical Seismic Deficiencies and Corrective Measures

- <u>Stewart Mountain Dam</u>: 212'-high thin arch (Phoenix, AZ)

  Deficiencies:

  – Upper portion of the arch could fail due to lack of bond across lift lines

  – Gravity sections and thrust blocks were determined to be unstable

  Modification:

  – Post-tensioned anchors were installed in the arch and thrust block

Typical Seismic Deficiencies and Corrective Measures

- <u>Pacoima Dam</u>: 372'-high arch dam (San Fernando, CA)

  Incident (1971 & 1994 Eqs):

  – Permanent slight tilting of dam crest and chord shortening of dam
  – Partial opening of contraction joints within the dam and between the dam and thrust block
  – Crack in left thrust block
  – Rearrangement and movement of rock mass

  Modifications:

  – Abutment stabilization by post-tensioned rock anchors; foundation curtain grouting, relief drains, thrust block crack repair and joint grouting